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LXXXII.

ON THE MECHANICAL PROPERTIES OF MATERIALS OF CONSTRUCTION,

AND ON VARIOUS PREVIOUSLY UNOBSERVED PHENOMENA, NOTICED DURING EXPERIMENTAL RESEARCHES WITH A NEW TESTING MACHINE, WITH AUTOGRAPHIC REGISTRY.

A paper by Prof. R. H. THURSTON, Member of the Society,

READ APRIL 4, 1874.

SECTION II.

14. INTRODUCTORY.—The preceding section has been devoted to a description of the peculiar form of testing machine employed by the writer during these researches, to an account of the general method of obtaining from any material an autographic record of its mechanical properties, and to the interpretation of strain diagrams thus obtained from the useful metals and other materials of construction.

The part here presented contains an account of some of the more interesting and fruitful of the special investigations conducted with the apparatus, and embodies a description of certain remarkable and hitherto unobserved phenomena accompanying the distortion of metals, the discovery of which must affect the theory of the effect of stress in producing strain, and, consequently, must somewhat modify engineering practice.

15. GENERAL DEDUCTIONS.—From what has already been shown, we may deduce the following *resumé* of methods of determining the several more important properties of materials by an inspection of their strain diagrams.

(1.) *To determine the homogeneousness of the material.*

Examine the form of the initial portion of the diagram between the starting point and the sudden change of direction which has been shown to indicate the elastic limit. Notice also its inclination from the vertical and compare with it the inclination of the "elasticity line."

A perfectly straight line beneath the elastic limit, perfectly parallel with the "elasticity line," shows the material to be *homogeneous as to strain; i. e.*, to be free from internal strains, such as are produced by irregular and rapid cooling or by working too cold. Any variation from this line indicates the existence, and measures the amount, of strain. A line considerably curved as in No. 6, Plate I, exhibits the existence of such strain.

Next, examine the form of the curve immediately after passing the elastic limit. A line rising from the elastic limit, regularly and smoothly, approximately parabolic in form and concave toward the base line, as in No. 22, indicates *homogeneousness in structure*, and the absence of such imperfections as are produced in wrought-iron by cinder, or in cast metals which have been worked from ingots, by porosity of the ingot.

A line turning the corner sharply, when passing the elastic limit, and then running nearly or quite horizontal, as in other irons, and in the low steel of Plates II and III, or actually becoming convex toward the base line, as with some of the woods in Plate I, and then, after a time, resuming upward movement by taking its proper parabolic path, indicates a decided want of this kind of homogeneity. The relative length of the depressed portion of the line, and the amount of depression, measures the relative defectiveness of materials compared in this respect.

Finally, compare the diagrams produced by several specimens of the same kind of material or from the same mass. Homogeneousness in general character and *homogeneousness in composition* are proven by the precise similarity of these diagrams, while a greater or less variation of the curves compared, indicates a greater or less difference in the specimens of which they are the autographs.

Materials should usually exhibit great homogeneousness in all these three ways, to be perfectly reliable. Perfect homogeneousness is not to be expected in either respect.

(2.) *To determine the elastic resistance of the specimen.*

Measure the height of the curve at the elastic limit, using the scale of torsion, or for tension, which is given for each machine and for each standard size of test-piece, as shown in the accompanying plates.

(3.) *To determine the resistance offered to any given amount of extension, or that producing a given set.*

Measure the ordinate of the curve at the point whose abscissa, or distance from the origin, measures the assumed degree of set.

(4.) *To determine the ultimate resistance of the material.*

Measure in a similar manner the maximum ordinate of the curve.

(5.) *To determine the resilience of the piece within the elastic limit; i. e.* the work required to produce an evident and permanent set, approximately proportional in amount to the degree of change of form of the specimen. This quantity measures the power of the material to resist blows, and its determination is evidently quite as important as that of resistance to static stress, which latter forms one of the factors of the former.

Measure the area comprised between the ordinate of the curve at the elastic limit and the initial part of the curve; this quantity is proportional to the required value. Or, multiply the elastic resistance of the material by the extension within the elastic limit. As an approximate result, two-thirds this product is the quantity required in inch-pounds or foot-pounds, according as measures of extension are taken in inches or feet.

(6.) *To determine the resilience of the material within any assumed limit of extension; i. e.*, the magnitude of blow required to produce a given set.

Measure the area of the curve up to the assumed limit, as for example, the area in Plate III, under No. 21, Z, 21, 21, Y, x, where the assumed set is the extension from Z to x. Two-thirds the product of the resistance measured by the altitude Yx, and the extension bx gives as before an approximate value for ordinary purposes.

(7.) *To determine the total resilience* or shock resisting power of the material.

Measure the total area of the diagram. For ductile materials an approximate value is obtained by taking two-thirds the product of the maximum tenacity by the maximum extension. For hard and very brittle materials one-half the same product gives very accurately its values. For intermediate qualities the true value is more nearly two-thirds this product, also Swedish wrought-iron, white cast-iron, and hardened steel illustrate the first and the second classes; ordinary tool steels are examples of the third class, as is also iron like No. 22.

(8.) *To determine the effect of a load given in pounds per square inch of stress.*

Find a point in the curve having an ordinate which measures the given stress. The abscissa of that point measures the extension under that load. In other words, a point being found in the curve, the height of which above the base line is equal to the load per square inch, its distance from the origin measures the extension of the material as produced by that stress.

(9.) *To determine the effect of a blow or a shock, whose measure is given in inch-pounds of energy, i. e.* of which the work that it is capable of doing is known.

Find a point on the curve whose ordinate cuts off an area between itself and the origin, representing the given amount of work. Or, find such a point that two-thirds the product of the stress measured by its ordinate, and the extension corresponding to its abscissa is equal to the number of inch-pounds given. The position of this point shows the maximum strain and the maximum extension of the material under the assumed conditions. Drawing a line through this point parallel to the nearest "elasticity line," the distance of the point at which it intersects the base line from the origin indicates the resulting set.

(10.) *To determine the effect of a blow upon the material, when already strained by a dead load.*

Determine first the extension produced by the application of the static stress, as in (8), and then find that point on the curve between the ordinate of which and the ordinate of the point indicating the strain just found as due the dead load, an area is intercepted which measures the work done by, or the energy of, the shock which has been assumed or calculated.

15. Examples.—Illustrations of the first seven of the above described processes are either given in the preceding portions of this paper, or will be noticed hereafter during the progress of special researches. Those succeeding may be illustrated thus:

(1.) Given, a load of 30,000 pounds per square inch. Determine its effect upon good qualities of cast and wrought iron, low steels, tool steel and the weaker metals.

Referring to Plate II for examples, we find that neither cast copper, lead, tin, nor zinc would sustain such strain; all would be broken.

Good irons, Nos. 1 and 6, would be strained beyond their limits of elasticity, and would take a set after an extension of about 1 and 1¼ per cent., respectively. The exceptional iron, No. 22, would be strained to a

point which is so nearly its elastic limit that it would remain practically uninjured.

The low steels, Nos. 69, 67, 76, would bear the stress with a similar degree of safety, very nearly. The first would have a considerable margin of safety within its elastic limit; No. 67 would be nearly, and No. 76 would be quite, strained to the elastic limit, while No. 98 would take a set of about one-fifth of one per cent.

If the strain were torsional, the weaker metals would be twisted off by a force corresponding to that here assumed, the good irons would take a set of about 25°, the best iron and the three stronger steels would take no appreciable set, and the softest of the latter would set at about 10°. In these cases the specimens are supposed of standard size. For other sizes the forces producing similar effects would vary as the cubes of the diameters.

(2.) Given, the magnitude of a shock, or blow, *e. g.* as equal to that due a weight of one ton, 2,000 pounds, falling one foot, the rod taking the strain being of one square inch area of section and one foot long. Determine the effect for each of the above-named materials.

The effect of this blow is equivalent to an expenditure of energy amounting to $2{,}000 \times 12 = 24{,}000$ inch-pounds.

The weaker materials, not possessing an ultimate resilience of this amount, would be broken.

Forged copper would be strained and would take a set after very nearly $12\frac{1}{2}$ per cent. of extension, since

$$0.12\tfrac{1}{2} \times 12 \times \frac{24{,}000 \times 2}{3} = 24{,}000$$

the work done by the blow being equilibrated by the product of two-thirds the resistance, noted at 110°, Plate II, into the extension. Perfect accuracy of figures may be insured by perfectly accurate measurements.

The specimen of iron No. 1 would be given an extension and set of nearly 0.068, since the resistance, under this amount of stretch, would be approximately 45,000 pounds per square inch, and the work during extension would be

$$0.068 \times 12 \times \frac{45{,}000 \times 2}{3} = 24{,}000 \text{ inch-pounds.}$$

The iron of special grade No. 22 would be elongated $0.058 = 0.69$ inch, as

$$0.069 \times 12 \times \frac{52{,}000 \times 2}{3} = 24{,}000, \text{ nearly.}$$

The same blow would produce on the rod, if made of such steel as No. 69, an extension of $0.0384 \times 12 = 0.461$ inches, estimated thus,

$$+ = 24{,}000 \div \tfrac{2}{3}.78{,}000 = 0.461,$$

it being found by "trial and error," that the extension 0.0384 developes a maximum resistance of 78,500 pounds per square inch.

It is evident that, where extreme accuracy is required, the curves should be transferred to a new scale in which the abscissas should be a scale of elongation instead of angular distortion, and the area should be carefully measured.* For the latter work an Amsler "Planimeter"† is useful.

(3.) Given, a bolt of dimensions as last assumed, strained with the effect of a load of 30,000 pounds, as in example (1), determine the effect of a blow of 24,000 inch-pounds energy, occurring while the bar is sustaining the static load.

The effect of the dead load, as already calculated, is to produce a strain upon the low steels, and upon iron like No. 22, which would keep them extended only a very minute fraction of their original length, this extension being, even with the latter material, but 0.05 of one per cent. The effect of the blow would be, practically, the same as has just been estimated for the unloaded bar.

Nos. 1 and 6 would be, as already shown, extended 1 and $1\frac{1}{2}$ per cent., respectively, by the simple load. The added effect of the blow would be to produce an additional extension and set of 0.0533 and 0.0555 respectively, since the mean resistance, during this extension, is

$$\frac{45{,}000 + 35{,}000}{2} \text{ and } \frac{42{,}000 + 30{,}000}{2}$$

respectively, and the extension must be,

$$24{,}000 \div \frac{45{,}000 + 30{,}000}{2} \div 12 = 0.0533 \text{ and}$$

$$24{,}000 \div \frac{42{,}000 + 30{,}000}{2} \div 12 = 0.0555$$

The bar is stretched in the first case 0.64 inch; in the second 0.666 inch, by the blow, if made of such iron as that of specimens Nos. 1 and 6.

It should be remarked here, that although the diagrams obtained from the various materials tested, give data from which to estimate their relative value in resisting shock, the absolute results of calculation, with no modification for varying rapidity of action, will be but approximate.

* See London "Engineer," Nov. 8th, 1872.

† It is evident that diagrams accurately representing tests made with the common testing machine afford similar facilities for solving these problems.

This is a consequence of the facts that the inertia of the body struck will affect the result, and that the actual resistance varies with the velocity of rupture. A rod which will sustain safely the blow of a heavy body, would yield readily under a blow of similar energy struck by a light weight moving with proportionally increased velocity. The mathematical investigation of this effect, which has not hitherto been noticed, remains to be given. It is only necessary to state here that a rod of uniform section, and homogeneous in structure, will be uniformly extended by a force slowly applied. A blow received at one end will extend it most at the portion nearest that end, and the more rapid the blow the more is its effect concentrated. It is possible to produce actual fracture at one end by a very rapid blow, and for rupture to become complete before the shock is felt at the opposite end. This action is seen daily in every workshop where pieces are broken from heavy masses by the blow of a hammer.

The effect of a blow depends, therefore, not only on the magnitude of that product of mass into height due its velocity which we call *vis viva*, but also upon the magnitude of the factors. It further follows that of two materials having equal tenacity and equal ductility, the one having greatest density will be most liable to fracture by impact.* This inference is confirmed by experience.

In general a rod should be somewhat larger at the end receiving the shock, and this enlargement should be greater as the blow is more rapid. Conversely, blows of equal energy are the most injurious when given by bodies of light weight moving at high speed. This difference is exaggerated by any cause which increases the density of the material. The variation of resistance with the rapidity of rupture will be considered more at length hereafter.

It is readily seen that we have here an explanation of the fact, that fracture produced by a quick blow is granular in character, while a steady pull brings out the "fibrous" texture of iron. In the former case the action is concentrated upon a cross section close to the point at which the blow is received; in the latter instance inertia acts less effectively in resisting the transmission of the rupturing force to other portions of the piece, and the drawing out process is permitted to take place.

16. Peculiar Problems sometimes present themselves in practice which cannot be solved by any published methods—why, it is difficult to say, but partly, it is probable, because of a deficiency of experimental

* *Mechanics' Mag.*, Dec., 1871, p. 492.

data, and partly because known authorities have not been led by actual experience in engineering practice to perceive the importance of their applications.

Of these the following is an example:

To determine the effect of a succession of stresses, whether static or dynamic, each of which strains the material beyond the original or the acquired limit of elasticity. An illustration of this action is given by the repeated bending, stretching, or other form of distortion by external force, of any material producing at each application a new set. The same case is illustrated by the gradual elongation of a rod by repeated blows, the energy of each of which exceeds the elastic resilience of the material.

Determine the elastic resilience of the material existing previous to the application of each stress, by taking the area comprised between two lines drawn through that point on the curve of the material chosen, whose abscissa represents the existing extension, one of which lines is an ordinate and the other of which is parallel to the nearest "elasticity line." This area represents the elastic resilience of the piece; *i. e.* a blow having an equivalent energy would leave the piece uninjured and without set. Deducting this amount from the energy of the given blow, the remainder of the work done by that blow is expended in producing set or extension, and may be determined as already described.

The effect of a simple force may be determined by deducting from the total distortion producted by each application of that force, the elastic range of the material.

It is thus readily ascertained, in either case, how much each application will add to the set, and how many applications will be required to produce rupture. It is here assumed that distortion within the elastic limit leavs the piece uninjured, however often it may be repeated. This assumption seems correct, *a priori*, and is well sustained by the valuable researches of Wohler* and others.†

The effect of repeated bending, or other form of strain, can thus be inferred from an examination of the strain diagram of the material, obtaining from a single experiment a determination hitherto only obtained by a long and tedious process of repeated distortion. Such investigations of the "fatigue" of metals are often of great importance.

17. The Effect of Time on Metals left under Strain.—The effect

* *Zeitschrift fur Bauwesen*, 1860; *Festigkeitversucke mit Eisen und Stahl*, Berlin; also Lond. *Engineering*, 1871.

† Fairbairn: *Civil Engineer and Architects' Journal*, Vols. XXIII, XXIV.

of stress is modified when metals are left under strain for considerable intervals of time. It had generally been supposed that this effect was to weaken the resistance whenever the material was left exposed to a strain exceeding the elastic limit.

This idea seems sustained by the experiments of M. Vicat, made at Paris about forty years ago.* He states that four wires were extended, respectively, by $\frac{1}{4}$, $\frac{1}{3}$, $\frac{1}{2}$ and $\frac{2}{3}$ their ultimate resistance, and their elongations were observed and recorded at intervals of one year. The relative extensions observed indicated a gradual lengthening of the three which were strained beyond the elastic limit, and that most strained finally broke, after sustaining $\frac{2}{3}$ its original ultimate breaking weight two years and nine months, the point of rupture being finally determined by the action of corrosion which had not been entirely prevented.

The several extensions were as follows:

No. 1,	sustaining $\frac{1}{4}$,	33 months	.....	0.000	per cent.
No. 2,	" $\frac{1}{3}$,	"	.....	0.275	"
No. 3,	" $\frac{1}{2}$,	"	.....	0.409	"
No. 4,	" $\frac{2}{3}$,	"	.....	0.613	"

The rate of extension was nearly proportional to the times, and the total extension to the forces. M. Vicat concludes that metal thus overstrained will ultimately break, and his paper has caused much uneasiness among members of the profession, as indicating a possibility of the ultimate failure of structures having originally an ample factor of safety.

The elegant and valuable researches, also, of H. Tresca, on the flow of solids,† and the illustrations of this action almost daily noticed by every engineer, seem to lend confirmation to the supposition of Vicat.

The experimental researches of Prof. Joseph Henry, on the viscosity of materials, and which proved the possibility of the coexistence of strong cohesive forces with great fluidity,‡ long ago proved, also, the possibility of a behavior in solids, under the action of great force, analogous to that noted in more fluid substances.

On the other hand, the researches of the writer, as described in the first section of this paper, indicating by the form of strain diagrams, that the progress of this flow was accompanied by increasing resistance, and the corroboratory evidence furnished by all carefully made experiments on tensile resistance, as those of King and Rodman, Kirkaldy and Styffe, made it appear extremely doubtful whether materials were really weak-

* *Annales de Chimie, et de Physique*, 1834; Tome 54; p. 35.

† *Sur l'Ecoulement des corps solides*; Paris, 1869–72. ‡ *Proc. Am. Phil. Society*, 1844.

ened by a continuance of any stress, not originally capable of producing incipient rupture.

18. To determine this point, a series of experiments was made, the general result of which was first formally announced in a note to this Society,* in which the first experiment, commenced during the session of the National Academy of Science at the Stevens Institute of Technology, was described, and in which the first deductions, since slightly modified by an extended investigation, were given. In Plate III, No. 16, is a *fac simile* of the strain diagram obtained at the first experiment.

A piece of iron, of a good quality of metal, but badly worked, as shown in the sketch already given in Section 1, was placed in the machine and strained considerably beyond its elastic limit. It was then left twenty-four hours under the strain thus produced at *A*, Plate III. At the end of this period, the pencil was found precisely as it was left, and not the slightest evidence of yielding was noted. The slight depression observed in many examples to be given, is produced by a slight compression of the wood used in blocking the machine at the beginning of the interval. No evidence of flow was therefore obtained.

Upon attempting to produce further change of form, however, the unexpected discovery was made that the test-piece had acquired an increased resisting power. The pencil, instead of following the general direction taken the day previous, rose, as seen in the diagram, until a resistance was indicated, *exceeding by nearly 30 per cent.* that shown before the specimen was left under strain. This resistance having been overcome, the piece yielded with a slightly decreasing resistance, and, after considerable additional distortion, was left at *B* twenty-four hours. The result of the second experiment is seen to be an increase of nearly 15 per cent., and a third trial at *C* gave a small, but still perceptible, gain also.

This singular phenomenon appeared so remarkable and so important that experiments were continued upon various grades of iron, and upon other metals, the greatest care being taken to avoid any possible source of error. Several strain diagrams are given illustrating some of these experiments.

No. 10 represents that of a piece of good iron which is far more homogeneous and better worked than No. 16.

No. 68 is a piece of "Siemens-Martin steel" which was left under

* *See Trans. Am. Soc. C. E.*, Nov. 1873; *Journal Franklin Institute*; March, 1874.

strain, at *A*, twenty-four hours, and at *B* an equal length of time. In the latter case, the applied force was wholly removed, at the end of twenty-four hours, before an attempt was made to produce further change of form. On renewing the strain the resistance is seen to have acquired an increased intensity very nearly absolutely equal to that shown at *A*, and relatively greater, a fact which will be found to aid in the determination of the real character of the phenomenon. A third experiment, at *C*, shows a repetition of this action, and a fourth, similar to that at *B*, in all except time—for in the last experiment the time was but a fraction of an hour—gave a similar result. In each case it is noticeable that a slight fall from the maximum attained, follows the yielding of the test-piece.

No. 33, malleable cast iron, No. 52, double shear steel, and No. 81, tool steel, all exhibit this same stiffening under prolonged strain.

No. 17, "homogeneous chrome iron," was subjected to experiment four times. At *A*, the effect is very marked, and the resistance to further change of shape continues to increase slowly until left at *B* for a second trial. The maximum attained at *B* is not sustained, as further distortion occurs, and, after a slight decrease, the specimen was again left, the pencil resting at *C*.

Next day, the increase of resistance was found less considerable than at the previous experiment, and the line, after passing a maximum a few degrees beyond, falls quite rapidly. Fearing that the metal was about to rupture completely, it was left once more at *D*, another day, after which time its behavior was similar to that on earlier trials. It fully regains the maximum power of resistance noted after the trial at *C*, and, before rupture, it even slightly exceeds it.

The hardest material experimented upon was the very hard chrome steel, No. 21. Left at *A* three days, the resistance at that degree of distortion was increased about 8 per cent., and, again at *B*, four days under strain gave a rise of nearly 4 per cent, after which a considerable rise occurred, in the ordinary manner, before rupture took place.

An interesting experiment was made with the best Swedish iron, a metal of such wonderful purity and ductility that one specimen, of standard size, was twisted nearly 600° before completely breaking off.

No. 101, Plate III, is the strain diagram of the specimen tested for the purpose of determining the effect of continued stress. Here, as seems frequently the case, a loss of ductility apparently accompanies the increase of resistance and the total resilience appears to be comparatively slightly altered.

This specimen was strained until the limit of elasticity was just passed and was then left at *A* one day. The result, with even the slight distortion of but 6″, producing an extension of a very minute amount, is similar to that before noticed, and the behavior here exhibited probably gives a clue to the causes of this peculiar action. After this trial, several others were made, and the metal is seen to have behaved in a manner precisely similar to the other grades of iron.

19. Reviewing all of the large number of experiments made since the discovery of this effect of continued strain, carefully comparing the curves obtained with each other, and with the diagrams obtained in the ordinary way, and, finally, making a comparison of the conclusions drawn from this research, with the results of the experimental work of other investigators, the writer has been led to the following, as the most probable explanation of this singular molecular phenomenon.

These strain diagrams are the *loci* of the successive limits of elasticity of metal, at successive positions of set.

The phenomenon here discovered is an elevation of the limit of elasticity by a continued strain. The cause is probably a gradual release of internal strain, occurring in a somewhat similar manner to that observed previously in cast iron in large masses, and less frequently, and generally in a less marked degree, in wrought-iron and other metals, which have been worked in large pieces, and in which such strain has been more or less reduced by a period of rest.* This loss of strength in large masses of wrought iron is stated, by Mallet, to amount frequently to one-half.†

20. The manner in which this reduction of internal strain occurs, by continued stress at the limit of elasticity, as here observed, may be readily conceived.

When the metal is thus strained, many sets of molecules are placed in positions in which they exert a maximum effect tending to produce molecular changes which may equalize the originally irregular distribution of inter-molecular stresses. After a time, the change actually takes place by "flow," and the resisting power of the piece becomes increased, and its limit of elasticity raised, simply because its forces are now no longer divided, and may act together in resisting external forces. The diagram itself exhibits the best evidence of the occurrence of flow, but it is also

* Compare London "Iron," Stability of Iron Structures, Feb., 1874; Van Nostrand's Magazine, April, 1874.

† "On the Coëfficients of Elasticity and Rupture in Wrought Iron in Relation to the Volume of the Mass, its metallurgic Treatment and the axial Direction of the constituent Crystals." *Proc. Inst., C. E.*

shown by an inspection of the broken specimen, as in Figures 6, 7 and 11, of Section I.

It was at first suspected that an action had been detected in this phenomenon, similar to that by which the "portative force" of a magnet is increased by loading its keeper more and more heavily until it becomes "supersaturated." It is not impossible, perhaps not improbable, that the two cases are similar, in some respects, this behavior of the cohesive force in the present example, aiding to produce the extraordinary increase of resisting power here observed.

A comparison of the ductility registered by samples variously treated lends some confirmation to the supposition, formerly expressed, that, in all cases, the resilience does not increase in the same proportion as the increase of mean resistance, as a consequence of sustained stress, and this, if a fact, may possibly be considered as corroborating the idea just suggested.*

The discovery of this elevation of the elastic limit in all metals examined is also probably confirmatory of this idea.

If the explanation, just offered, of the apparent strengthening action of prolonged stress is correct, the conclusions of Vicat are not overthrown, although evidently not fully justified by his own experiments, and although the intervening forty years of engineering practice have not produced evidence which may be considered as confirmatory of them.

The same molecular movement, or flow, which rearranges the internal forces and relieves internal strain, may be a phase of that viscosity which Vicat supposed might in time permit rupture of metal, subjected to stress nearly approaching its original ultimate resistance, the one action being a more immediate result than the other, and the latter producing its effect, even when cohesive force may have been actually intensified.

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* Since the above was written, the *Journal of the Franklin Institute* has, in the issue for March, 1874, given an account of experiments made by Com'd'r Beardslee, U. S. N., during which metal strained beyond the elastic limit by tension, exhibited a gain of resistance at a position of earliest set, *i.e.*, an elevation of the elastic limit, from 23,075 to 26,100 pounds per square inch, 13.1 per cent., in seventeen hours. The material was bloom iron turned approximately to one-half square inch section.

A very important fact noted by the experimenter is that an apparent positive action was observed in the metal under strain by which the scale beam was actually thrown up by a force measuring 125 pounds. If this observation is not erroneous, or, if the action was not due to some accidental circumstance, we may have here a measure of the *intensifying effect* above described.

Commander Beardslee has communicated to the writer the experimental confirmation of the fact, deducible from the explanation here given, that this release of internal strain occurs to a nearly equal extent if the strained piece is simply laid aside for a similar interval after it has been given a set.

Should this prove to be the fact, it would seem allowable to conclude that the forces of polarity and cohesion are not identical. The cause of the apparent increase of strength of iron, with increase of temperature, is seen to be explained by this relief of internal strain, which occurs most readily at high temperatures.

The experiments of the writer have not indicated the possibility of continued flow, and, consequently of ultimate rupture, except under stress, increasing in intensity up to the full maximum resistance of the material.* They do not, therefore, confirm Vicat's deductions, and the inference would seem, on the contrary, to be—that structures of metal do not become weakened with age, except as injury occurs by corrosion, or by overloading. The experiments of Roebling† and his opinion, as expressed in his report on the Niagara Supension Bridge, are apparently correct.

Kirkaldy, also, concludes that the additional time occupied in testing certain specimens of which he determined the elongation "had no injurious effect in lessening the amount of breaking strain."‡ An examination of his tables shows those bars which were longest under strain to have had highest average resistance.

21. Wertheim supposed that greater resistance was offered to rapidly, than to slowly, produced rupture; Kirkaldy concludes that the contrary is the case. Redtenbacher§ and Weisbach‖ assume the law of resistance to be the same beyond the limit of elasticity as within it, and deduce formulas for resistance to shock which are widely inaccurate.

The experiments of the writer prove that, as had already been indicated by Kirkaldy (whose results, however, had been looked on by many of the profession with some suspicion), a lower resistance is offered as the stress is more rapidly applied. This conspires with *vis viva* to produce rupture.

This is seen at *w*, No. 101, Plate III, where a sudden increase of velocity produced a depression of the line, at the angle 110°, and it is exhibited in a much more marked degree in No. 118.

In the latter example, the strain was gradually applied until the point *a* was reached, when, with a suddenly applied force, a motion estimated at about one-tenth of a foot per second, was obtained and, immediately at *b*, the resistance fell off very considerably, the pencil dropping to *c*.

* Compare Kirkaldy—*Experiments on Wrought Iron and Steel*—pp. 62-69; also see the plates given by Styffe, exhibiting curves for tension.

† *Journal Franklin Institute*, 1860; Vol. XL, p. 360.

‡ *Experiments on Wrought Iron and Steel*, pp. 62, 83.

§ *Der Maschinenbau*, Vol. I.

‖ *Mechanics of Engineering, etc.*

Again resuming the slowest movement, about one hundredth of a foot or less per second, resistance rose again to *d*. A repetition of the rapid movement between *d* and b^1, was followed by a loss of resistance again from b^1 to c^1, and, as is seen by the diagram, this occurred whenever the experiment was repeated. At *k*, distortion by a very slow movement was resumed, and continued until the specimen broke. Here we have probably, the first direct determination of this question, in which the effect of *vis viva* does not appear.

We may, therefore, conclude that the rapidity of action, in cases of shock, and where materials sustain live loads, is a very important element in the determination of their resisting power not only for the reason given already in paragraph 3 of this section, but because *the more rapidly the metal is ruptured, the less is its resistance to rupture.* This loss of resistance is about 15 per cent.* in No. 118.

The cause of this action we may presume to bear a close relation to that operating to produce the opposite phenomenon of the elevation of the elastic limit by prolonged stress, and it may probably be simply another illustration of the effect of internal strain.

With a very slow distortion, the "flow" already described occurs, and but a small amount of internal strain may be produced, since, by the action noticed when left at rest, this strain relieves itself as rapidly as produced. A more rapid distortion produces internal strain more rapidly than relief can take place, and the more quickly it occurs, the less thoroughly can it be relieved, and the more is the total resistance of the piece reduced. Evidence confirmatory of this explanation is found in the fact that bodies most homogeneous as to strain exhibited the least of these effects.

It does not seem impossible that, at extremely high velocities, the most ductile substances may exhibit similar behavior, when fractured by shock, or by a suddenly applied force, to substances which are really comparatively brittle.

In the production of this effect, which has been frequently observed in the fracture of iron, although the cause has not been recognized, the inertia of the mass attacked and the actual depreciation of resisting power just observed, conspire to produce results which would seem quite inexplicable, except for the evidently great concentration of energy here referred to which, in consequence of this conspiring of inertia and reduced

* Compare Kirkaldy, p. 83, where experiments, which are possibly affected by the action of *vis viva* indicate a very similar effect.

resistance, brings the total effort upon a comparatively limited portion of the material, producing the short fracture with its granular surfaces, which is the well known characteristic of sudden rupture.

Any cause acting to produce increased density, as reduction of temperature, evidently must intensify this action of suddenly applied stress.

The liability of machinery and structures to injury by shock is thus greatly increased, and it is quite uncertain what is the proper factor of safety to adopt in cases in which the shocks are rapidly produced. This uncertainty must remain until further experiment shall be made the basis of a correct mathematical expression of the natural laws involved in the problem.

Meantime the precautions to be taken by the engineer are: To prevent the occurrence of shock as far as possible, and to use in parts exposed to shock, light and elastic members, composed of the most ductile materials available, giving them such forms as shall distribute the distortion as uniformly and as widely as possible.

22. The behavior of materials subjected to sudden strain is thus seen to be so considerably modified by both internal and external conditions, which are themselves variable in character, that it may still prove quite difficult to obtain mathematical expressions for the laws governing them. It is not improbable, however, that an approximation, of sufficient accuracy for all cases which frequently arise in practice, may be obtained by a study and comparison of experimental results obtained, as above, by the method here adopted, and one which seems peculiarly adopted to the work.

A carefully conducted series of experiments giving quantitative results would be of great value. Without such a research no reliable knowledge can be obtained of the law of depreciation, and no useful formulas can be devised for use in calculation. The experiments made by the writer are not yet sufficiently numerous or precise to serve as data from which to deduce equations.

23. The Elasticity of the Metals.—The examination of the "elasticity line" will be found to present some facts of interest.

It will be seen that, in every case, the line produced by the descent of the pencil is not precisely coincident with that formed by its rise. This is not due to the friction of the machine, as that would not cause the decided difference observable in the form of the curve. It may be partly due to the fact that the set produced is partly temporary.*

* Morin, *Resistance des Materiaux*, p. 10.

An attempt was made to determine its law by the following method:

A steam gauge having a recording apparatus, in which the paper was moved horizontally, at a uniform rate, by a well-constructed clock, was kindly furnished by the Messrs. Edson, of the New York Recording Gauge Co.

This was set up by the side of the machine, upon a stand on which could be clamped the test piece. The latter carried a light, long pencil holder, so arranged as to traverse the paper in a direction at right angles to that of the motion given it by the clock. An angular motion in the specimen of 0.05° would be readily observed. The specimen was given a degree of torsion of from 10° to 360°, in different cases, and was then rapidly transferred to the stand, where the restoration of form would record itself upon the moving paper, forming a curve of which the ordinates would represent the restoration of form, and the abscissas would be proportional to the time.

In all cases observed, the restoration of form, by loss of temporary set, was so rapid as to have become completed before the specimen could be placed in the recording apparatus, and the record made a straight line invariably.

The conclusion which has finally been deduced from a study of the diagrams obtained is, that the peculiar feature here alluded to is a consequence of an internal molecular friction, the existence of which has already been long suspected by the writer, and probably by many other experimenters.

An illustration of a similar action probably occurs in the behavior of iron under magnetic action. Magnetization produced in bars of various qualities of iron and steel during important researches of Dr. Joule* and Prof. Tyndal,† proved this behavior to be common to all, when the change of form was produced, within a very minute range even, by magnetic force. Dr. Mayer has more recently ‡ examined this peculiar form of molecular action with great skill and thoroughness. Its existence is undoubtedly well proven, and the lines, above referred to, on the strain-diagrams seem to exhibit its effect very clearly. Possibly the rise from *f* to *g*, No. 118, Plate III, is due to such friction.

24. The evident proof found, in the parallelism of all elasticity lines in each diagram, of the fact, first noted by some of the earliest experi-

* *Philosophical Magazine;* 1874. †*Researches on Diamagnetism*, etc., 1870.

‡ "*Effects of Magnetism in changing dimensions of iron and steel bars,*" by A. M. Mayer, Ph. D.; Steven's Institute of Technology; 1872. *American Journal of Science and Arts;* 1873.

menters in this field, that elasticity remains quite unimpaired up to the point at which rupture commences, has been already adverted to.

Coulomb describes a series of curious and instructive experiments which may assist in determining the molecular action occurring in these instances where great distortion and great permanent displacement of particles takes place without loss of elasticity.*

He found this to occur, not only with metal wires, but with threads of fine clay, $\frac{1}{12}$ of an inch in diameter and 11 feet long, which could be twisted $5\frac{1}{2}$ turns repeatedly without set and without apparent loss of elasticity. Turning the thread through a wider range of torsion, it always returned but $5\frac{1}{2}$ revolutions, and in each new position of set exhibited the same elasticity as before.

The explanation of this action, as illustrated by the strain diagrams of Plates II and III, and by Coulomb's experiments, is probably also to be found in the phenomenon of flow of solids. The restoration of cohesion, in bodies actually separated, exhibits the extent to which this action may proceed. Two freshly cut surfaces of lead when brought together, with a moderate pressure cohere firmly, and plates of glass, laid one upon another, sometimes "seize" each other so firmly that they are cut and worked as one piece.† The welding of iron is another and a very familiar illustration of the same action. Cohesion may therefore be actually destroyed and renewed, and molecules may move among each other, changing completely their relative positions, without loss of either strength or elasticity.

The result of these experiments on metal are important as exhibiting the error of the opinion hitherto entertained by many physicists and engineers, among whom was the writer,‡ that straining metal might weaken it, even when rupture did not commence, and even where no condition of internal strain was induced. It has been here shown that elasticity remains unimpaired, and resistance continually increases up to the point at which incipient fracture takes place. No well proven exception to this law has been observed.

25. While comparing the inclination of the elasticity lines with the initial line, to determine the pressure and the amount of internal strain, it has been noticed that more or less strain seems almost invariably to exist, but that the amount, as indicated by the difference in inclination of

* *Lecture Notes on Physics;* Mayer. *Journal Franklin Institute,* 1868.
† *Miller: Chemical Physics;* p. 67.
‡ *Journal Franklin Institute.*

the two lines, is not always as well shown by the greater or less curvature of the initial portion of the diagram. The probable reason would seem to be that this strain is not always uniformly distributed. Were the strain considerable and uniformly distributed, the initial line would be strongly convex toward the base line, and would have the parabolic form. Absence of strain is indicated by a straight line rising regularly to the elastic limit, or, in many cases where the elastic limit is very low, and when the material is inelastic and flows without tendency to recoil, concave toward the base, and parabolic. Irregularly distributed strain would modify the parabolic curve, and the amount of strain would determine the total curvature.

The initial and elasticity lines have, therefore, great interest as revealing important and otherwise unrecognizable properties of the material.

It has been remarked that the difference of inclination just referred to, proves the truth of the assertion of Hodgkinson that every load produces a set. It can now be readily seen why this should usually be the fact, and also that, although it is true, it does not necessarily indicate injury of the material.

Since, in its ordinary state, many sets of particles are usually in a condition of maximum strain, the slightest application of external force to the piece will destroy the existing equilibrium among these conflicting forces within the mass, producing a change of form, and either rupturing or producing a flow of those particles which are most strained, and thus causing a new condition of equilibrium, the piece returning only approximately to the original form when relieved. The greater or less the applied force, the greater or less the number of displaced particles, but it is only when the set becomes nearly proportional to the distortion that it assumes the character in which it is looked upon as a serious effect.

With perfectly homogeneous materials, free from internal strain, no such action would be noticed, and the earliest set would occur beyond the elastic limit, which limit is here considered to be attained when the set becomes proportional to the distortion.

26. The very minute range of distortion within the elastic limit is shown by the strain-diagrams very beautifully. This point is usually reached with the first 5° of torsion, and where internal strain has been eliminated it is frequently found within 2°, the corresponding extension being much less than .0001.

Captain Rodman, who has made the most delicate determination yet published,* detects a set of 0.000,001,4 after an extension of 0.000,27 in a specimen of cast-iron, and the elastic limit, as here defined, is reached after an elongation of about 0.0003, the point not being, however, very accurately determinable on account of the insensible change of rate, as already observed in the strain-diagrams of cast-iron.

The immense magnification, on the strain-diagrams, obtained with the torsion machine, of the elongation at the commencement of the curve enables the behavior of the materials within this minute, yet most important, portion of the entire range to be perfectly represented, and permits its examination in a most satisfactory manner.

27. The Influence of Variations of Temperature. — The effect upon the mechanical properties of metal of variations of temperature has long been a subject of debate, and one which has not even yet been satisfactorily settled by experiment.

A priori it would appear that, in a perfectly homogeneous material, entirely free from internal strain, change of temperature would produce an alteration of strength and of ductility which would both be the reverse, in direction, of the variation of temperature.

The forces acting to produce mechanical changes being, probably, cohesive force, on the one hand, resisting external forces tending to produce distortion or rupture, while the force produced by the energy of heat motion conspires with external force to produce that distortion, and the molecules being, at every instant, in equilibrium between the force of cohesion, on the one side, and the sum of the other two forces mentioned, on the other, variations of form must ensue with every change in the relative magnitudes of these forces. A change of temperature produced by an increment of heat energy, it would appear, must produce a reduction of cohesion by separation of particles, and the opposite change must cause an increase of cohesion by their approximation. Increase of temperature, by reducing the range of action of cohesion by separating particles, and causing them to approach the limit of reach of cohesive force, would reduce ductility, and the opposite change of temperature would increase extensibility. The effect on resilience, the product of ductility and strength, would evidently be still more marked than the variation of its factors.

The peculiar behavior of zinc, and the often observed brittleness of

* *Experiments on Metals for Cannon*, etc., Rodman, pp. 157-167.

iron, at low temperatures, have given cause for doubting the truth of the above statement, and until the phenomena accompanying variations in homogeneousness of structure and composition, and the introduction or removal of internal strain, have been very thoroughly investigated, it cannot be anticipated that the subject will become well understood. The character of polarity, that force of which the presence constitutes the distinguishing difference between solids and liquids, remains to be determined, and its determination may be expected to throw important light upon this subject.

Experiments of both physicists and engineers have failed, up to the present time, to give as much, and as precise information as is needed to determine satisfactorily what rules should govern the proportions of structures, whether carrying dead loads or subjected to shocks or blows, at any given temperature below the usual range, or even at the low temperatures to be met during every winter in the latitude of New York.

In a paper recently published* on "Molecular Changes produced by Variations of Temperature," the writer gave the results of a careful investigation of the experimental work previously done, by both philosophers and engineers, in researches bearing upon this important question.

28. The conclusions, as there reached, were the following:

(1.) That the number and the nature of those molecular forces which determine the physical condition of matter are not yet fully ascertained, but that these forces manifest themselves in, at least, three distinct modes of action, and, as thus exhibited, they are known as repulsion, cohesion, and polarity.

(2.) That the force of repulsion is, apparently, heat motion, or some closely related phase of energy. That the force of cohesion bears some resemblance to that of gravitation, but seems not to be identical with the latter, and that the force of molecular polarity, which determines the molecular relations of position, seems to bear some distant relation to that of magnetic polarity.

(3.) That the law which governs the variations in intensity of these forces with changes of intermolecular distances, is undetermined, and that it has not been expressed by any mathematical formula, except approximately and for a limited range.

(4.) That the magnitudes of the intermolecular spaces, and, conse-

* *Iron Age*, June, 1873; *Van Nostrand's Mag.*, July, 1873; *Jour. Frank. Inst.*, 1873; *London Jour.*, Jan., 1874. Also in pamphlet, p. 29. D. Van Nostrand.

quently, the volume of any mass, are variable with changes in the relative magnitudes of the forces of cohesion and repulsion.

(5.) That the resistance offered to change of form is determined by the relations in intensity of the forces of polarity and those forces which determine intermolecular distances.

(6.) That, at the "absolute zero" (—461.2° Fahr.), cohesion, and consequently the strength, of the material has, probably, its maximum value, heat-energy having disappeared.

(7.) That, at very high temperatures, heat-energy exerts a separating force upon particles, which entirely overcomes the other forces, and matter, assuming the gaseous state, requires the action of extraneous forces to preserve its volume unchanged.

(8.) That, at intermediate points, matter, in either the solid or the liquid states, exhibits a definite degree of separation of molecules which is determined by the intensity of the repulsion due to heat motion, a position of equilibrium being assumed, which, with the same substance, is invariable for the same temperature. The application of some kind of force is required to disturb this equilibrium and to produce change of volume. The amount of this force is determined, for any given extent of disturbance, by the maximum value of cohesion for the substance and the quantity of heat which has been required to raise it from the absolute zero of temperature.

The sum of the applied force and the force consequent upon the presence of heat motion must exceed cohesive force to produce dilatation, while this cohesive force, added to externally applied force, must exceed the force of repulsion to produce diminution of volume.

(9.) That the distinction between the solid and liquid states of matter seems due to the action, in the former, of the force of polarity, which gives stability of form, while, in the latter, this force is extremely feeble, disappearing altogether when the boundary line between the liquid and gaseous states is reached.

That combined stability and elasticity of volume may be produced by the equilibrium of attractive and repulsive forces, but that stability and elasticity of form demand the coexistence of cohesion and polarity.

(10.) That the general effect of increase or decrease of temperature is, with solid bodies, to decrease or increase their power of resistance to rupture, or to change of form, and their capability of sustaining "dead" loads.

(11.) That the general effect of change of temperature is to produce change of ductility, and, consequently, change of resilience, or power of resisting shocks and of carrying "live" loads. This change is usually opposite in direction and greater in degree than the variation simultaneously occurring in tenacity.

(12.) That marked exceptions to this general law have been noted, but that it seems invariably the fact that, wherever an exception is observed in the influence upon tenacity, an exception may also be detected in the effect upon resilience. Causes which produce increase of strength seem also to produce a simultaneous decrease of ductility, and *vice versa*.

(13.) That experiments upon copper, so far as they have been carried, indicate that, (as to tenacity), the general law holds good with that metal.

(14.) That iron exhibits marked deviations from the law between ordinary temperatures and a point somewhere between 500° and 600° Fahr., the strength increasing between these limits to the extent of about 15 per cent. with good iron. The variation becomes more marked and the results more irregular as the metal is more impure.

(15.) That above 600° F. and below 70°, the general law holds good for iron, its tenacity increasing with diminishing temperature below the latter point, at the rate of from 0.02 to 0.03 per cent. for each degree Fahrenheit, while its resilience decreases in a higher but not well determined ratio for good iron, and to the extent of reduction to one-third its ordinary value, or less, at 10° F. when cold short, and, in the latter case, the set may be less than one-fourth that noted at a temperature of 84° Fahr.

(16.) That the viscosity, ductility and resilience of metals are determined by identical conditions, and that the fracture of iron at low temperatures has, accordingly, been found to be characteristic of a brittle material, while, at higher temperatures, it exhibits the appearance peculiar to ductile and somewhat viscous substances. The metal breaks in the first case, with slight permanent set and a short granular fracture, and in the latter with frequently a considerable set and a form of fracture indicating great ductility. The variation in the behavior of iron, as it approaches a welding heat, illustrates the latter condition in the most complete manner.

(17.) That the precise action of the elements with which iron is liable to be contaminated, and the extent to which they modify its behavior under varying temperature, remain to be fully investigated, but that the presence of phosphorus, and of other substances producing "cold short-

ness," exaggerates, to a great degree, the effects of low temperature in producing loss of toughness and resilience.

(18.) That the modifications of the general law with other metals than iron and copper, and in the case of alloys, have not been studied and are entirely unknown.

(19.) That these conclusions are sustained by experiments of both physicists and engineers.

"The practical result of the whole investigation is that iron and copper, and probably other metals, do not lose their power of sustaining 'dead' loads at low temperatures, but that they do lose to a very serious extent their power of sustaining shocks or of resisting sharp blows, and that the factors of safety, in structures need not to be increased in the former case, where exposure to severe cold is apprehended, but that machinery, rails and other constructions which are to resist shocks, should have larger factors of safety, and should be most carefully protected, if possible, from extreme temperatures."

29. The conclusions above given were deduced from the physical investigations of Boscovitch, Coulomb, Henry, Powell, Cagniard de la Tour, Andrews, Faraday, Wartman, Robison, Gaudin, Thompson, Rankine and others, and from the more purely technical researches of Professors Johnson and Norton, Fairbairn, Kirkaldy, Brockbank, Joule, Spence, Styffe and Sandberg.

An apparent discrepancy of results from which some experiments seemed to indicate weakness, and others strength of metals as a consequence of reduced temperature, was explained by the fact that those which seemed to prove the first conclusion, were cases in which the metal was tested by blows, and those proving the reverse were tests made by steady strain. The deduction (11), made above, that the result of change of temperature, by producing changes of ductility, which were the reverse of those produced in tenacity, and that the same bar might thus exhibit a higher resistance to static stress while less capable of resisting shock, explained the seeming contradiction.

30. It was evident that, to determine satisfactorily the real effect of changes of temperature, it was necessary to obtain a series of experimental determinations of the simultaneous action of such variations upon both strength and resilience. Such experiments could readily be made by the method here pursued, and a considerable number of observations are represented by strain-diagrams on Plate III.

In these experiments, the machine and the test pieces were placed in

the open air where, changing in temperature with the atmospheric changes, no error could arise by transfer of heat during the experiments. The machine and the specimens submitted to test were always of the same temperature.

The mildness of the past winter has precluded the determination of the behavior of iron at temperatures very far below the freezing point, the lowest reached being 10° Fahr.

This is the more to be regretted since, as will be seen, there exists a possible change of law near the Fahrenheit zero, and it is extremely important to ascertain whether this indication of an anomaly arises from irregularity in the quality of specimens, nominally of the same grade, or whether it is a real variation of the effect of change of temperature.

As no previously made experiments combine, in the manner here presented, the various effects of heat upon the mechanical properties of the metals, the results obtained are given as a beginning, and the conclusions which are deduced from them are given as merely probable, while it is to be hoped that other members of the profession, who may be so situated that they can readily continue the work at points in the north and northwest where a temperature far below zero is reached, will make more complete and instructive researches during succeeding winters.

It is apparently quite impossible to avoid error if the attempt is made to experiment with specimens cooled down by freezing mixtures, and the writer would only feel justified in presenting the results of out-of-door work.

31. Referring to Plate III, the strain-diagrams of the best, and of medium tool steel, of German and double shear, of the several grades of iron and of copper and bronze are given, for temperatures from 70° down to 10° Fahr. A diagram is also given in which the horizontal scale of the plate is taken to represent absolute temperatures on a one-fourth scale, and at ordinates representing respectively 10°, 18°, 25° and 70°, the resistances of the several specimens are laid off, and dotted lines connecting them indicate the rate of variation of strength with temperature.

It will be seen that with the single exception of a scrap gray cast iron, presumably unusually impure (Nos. 25, 26), the effect seems invariably to have been a simultaneous increase of both strength and ductility with decrease of temperature to 18° and usually to 10° from 70° Fahr. In the case of the cast-iron, the increase of tenacity and reduction of ductility at the lower temperature are equally well exhibited, and the result is a slight decrease of resilience.

It will be noticed that the general trend of the lines in the diagram prepared for comparison of results, is very evidently towards a point on the scale (250°), corresponding to a temperature of 1000° above the absolute zero, at which point, were the diminutions proportioned to temperature, the metals would lose all cohesion. Since, however, the law, as determined approximately by the Committee of the Franklin Institute, is expressed by a parabolic equation, the fact that their melting points are nearer 3,000°, or perhaps 4,000°, Fahr. above the absolute zero, does not conflict with the results of experiment.

32. Comparing the several specimens of "good cast-steel," it is found that the four (Nos. 46, 47, 49, 50), whose strain-diagrams are given, evidently differ nearly as much in their individual properties as in their alteration by temperature. The two pieces tested at 70° Fahr. give a mean of strength, ductility and resilience, which is less than either of the other specimens. The strongest piece was broken at 18°, while that tested at 10° is very nearly its equal in that respect, and is more than 10 per cent. better in extensibility and nearly 10 per cent. superior in resilience. The difference between the specimens tried at 10° and at 70° (average of the two) is about 15 per cent. in ductility and resilience, and rather more than 5 per cent. in tenacity. The piece tested at 10° has a limit of elasticity exceeding that of the specimens tested at 70° in about the same proportion.

The double shear steels are irregular, as would be expected from their method of production, but the greatest ductility is shown by the specimen tested at 25° F., and the greatest tenacity by that broken at 25° also. The weakest is that tested at 70°, its loss of strength, ductility and resilience being very striking. The position of the elastic limit varied with that of the ultimate strength. "German" (English) steel exhibits greatest strength at 18° (No. 60), greatest ductility at 70° (No. 58) and greatest resilience at 25° (No. 25).

"Medium crucible" steels seems strongest at 18° Fahr., most ductile and equally resilient at 25°, and weakest, least ductile and least resilient at 70°, Nos. 78, 54, 70 are their strain-diagrams.

Swedish irons (No. 99 and 100) were tried at 25° and 70°, and the result is again that the greatest resistance and greatest extensibility occur at the lowest temperature, the difference here amounting to something less than 10 per cent. at the elastic limit and about one-half as much at the maximum.

A piece of common iron was selected by the blacksmith from his stock as "the worst specimen of cold short iron in the shop." The two specimens, Nos. 130 and 132, were taken from the bar and tested at 10° and 70° Fahr. respectively, with a result which is unexpectedly similar to those already given in the variation of ultimate strength, ductility and resilience. The increase of strength at the lower temperature is apparently nearly 15 per cent. the increase of ductility about the same, and the increase of resilience 30 per cent. At the elastic limit this is reversed, however, the specimen tested at 70° showing the highest elastic resistance. That broken at 10° exhibits a very considerable amount of internal strain, to the presence of which may be attributed the exceptional behavior of these pieces. A similar difference but opposite in direction and less in amount is noticeable in the Swedish irons, Nos. 99 and 100.

Copper, Nos. 133 and 134, and bronze, Nos. 137, 138, both show a considerably greater strength at 10° than at 70° and a slightly improved ductility. The increase of tenacity and resilience amounts to 20 per cent. and 30 per cent. respectively, the ratio being slightly greater at the limit of elasticity.

Cast-iron exhibits the most striking increase of tenacity, the ratio of increase being above 30 per cent. and in ductility having a mean value of 50 per cent. One specimen, No. 25 *C*, has a serious defect of internal strain, but eliminating that by cutting off the sharply curved lower portion of the line, it would coincide very exactly with the companion specimen, No. 25 *D*, broken at the same temperature, 25°.

33. It would seem, after a study of these experiments and after a comparison with those described by other experimenters, that, although a considerable irregularity, due to differences in material nominally of identical character, tends to obscure them, that we may feel some confidence in drawing the following conclusions in modification and extension of those already given in article 28:

(20.) That, with pure, well-worked metals, the principle enunciated in article 28 is fully illustrated, and a decrease of temperature is accompanied by increase of strength, ductility and resilience.

(21.) That materials which are impure and irregular in character may exhibit exceptions to, and even reversals of, that principle in changes of ductility and, while increasing in power of resisting simple stress, may lose their power of resisting shock, by a diminution of temperature.

We may hence feel confidence that, with really good iron or steel, we

are not exposed to seriously increased danger of failure of structures at such low temperatures as frequently occur in this latitude.

There is no reason to believe that the familiar effect of phosphorus in "cold-shortening" iron and steel, when cold, has its maximum effect at ordinary temperatures. The experiments of Sandberg would seem to prove this effect to be intensified continually with decrease of temperature.

34. Since the above research was concluded, the writer has become indebted to the thoughtful kindness of Mr. Chas. Francis Adams, Jr., for reports, one of which is that of the Mass. R. R. Commissioners for 1874 containing (page 144, *et seq.*), the result of an investigation of the cause of rail breakage on a considerable number of railroads in the United States and Canada. The conclusions given are that "cold does not make iron or steel brittle, or unreliable, for mechanical purposes," and that "it was not the rule that the most breakages occur on the coldest days." The introduction of steel, in place of iron rails has caused an almost complete cessation of the breakage of rails (p. 150).

The deduction which would seem proper in comparing these latter statements with the work of Sandberg would seem to be, simply, that the latter experimented with cold-short rails. The same conclusion is given in Sandberg's own statement of the difference between Welsh and French metal.*

35. We still require, to give us reliable information regarding exceptional cases, a series of experiments to determine the action of excessively low temperature, and whether the apparent change of law near the Fahrenheit zero is a natural or an artificial phenomenon. We need to learn precisely the effects of sulphur, phosphorus, and silicon at extremely low temperatures. We need, also, and especially, to learn by experiment whether extremely low temperatures occurring during our winters, produce a serious effect upon iron and steel by the introduction of internal strain as the material decreases in volume and increases in density.

The uncertainty still existing as to the extent to which increased density at low temperatures and reduction of tenacity under sudden strains at all temperatures—phenomena which have been revealed but not measured during the investigations here described—will be recognized by every member of our profession as one which it is exceedingly important

* P. 157; conclusion 3; p. 158, line 4; p. 132, Nos. 21, 24.

to remove. It is hoped that it may not long remain, for it is evident that, although perhaps improbable, it is not impossible that, notwithstanding the increase of both tenacity and ductility by reduction of temperature, these causes may still conspire to produce increased tendency to yield before shock at some unknown low temperature.

36. The peculiarities of fracture, which have been alluded to, are exhibited in the accompanying Plate.

Figures 15 and 16 exhibit the behavior of a bar of iron made at Catasauqua, Pennsylvania, as broken by Mr. Oliver Williams, at 70°, and, again, at 20° Fahr. It is selected from among the specimens in the cabinets of the Stevens Institute of Technology. The fracture at 70° (Figure 15) has the fibrous fracture and all the characteristics of what is generally considered a tough and ductile iron. That at 20° (Figure 16) resembles the break produced by a quick blow in good iron, or by any treatment at ordinary temperatures of a cold short iron. Were the conditions stated not known it would be supposed that irons like Nos. 1 and 16, Figures 5 and 6, were combined in a single bar.

Figures 17 and 18 illustrate strikingly the difference between the specimens of copper, Nos. 87 and 133, which have been already described and of which the strain-diagram of the one is shown in Plate II, and of the other in Plate III. No. 87 was cast in green sand, and broken at 70° F., while the other was made from the same ingot, but cast in dry sand and broken at 10° Fahr.

The first is unsound in structure in consequence of the dampness of the mould and exhibits a peculiar radiated texture which is probably due to the same cause. The second is distinguished by its compact, homogeneous structure, probably due to the freedom of the mould from vapor and gases. It presents a beautiful crystalline fracture which is probably due partially, if not principally, to the low temperature at which it was broken.

These fractures are extremely interesting from the strongly typical features which they exhibit as characteristic of the peculiar conditions under which they were produced.

Résumé.

37. *En resume*, a review of this investigation of the nature of, and the influences affecting, the distortion and rupture of metals, it would seem allowable to accept, as extremely probable, the following

General Conlusions:

(1.) That accurate strain-diagrams, in which the behavior of the material, as distortion progresses, and especially about the elastic limit, afford a means of acquiring valuable information respecting the strength, elasticity, homogeneousness, ductility and resilience of materials, and of tracing the modifications induced by variations of treatment and of composition.

(2.) That internal strain plays a most important part in determining the behavior of materials strained by either static or dynamic stress.

(3.) That the time, during which applied stress acts, is an important element in determining its effect, not only as an element which modifies the effect of the *vis viva* of the attacking force and the action of inertia of the piece attacked, but, also, as modifying seriously the conditions of production and relief of internal strain by even simple stresses.

(4.) That with good materials, cold does not produce injury but actually improves their power of resisting stress and increases their resilience.

(6.) That the influence of impurities, of various methods of manufacture, of changes of density with temperature, and of the causes which produce a concentration of the action of rapidly produced distortion and of quick blows, are subjects which still require careful investigation.

(7.) That experiment confirms the theory as to the behavior of materials, homogeneous in composition, structure and strain, as expressed *a priori* in 27, and hence a probable deduction that the force of molecular repulsion is heat motion.

Fig. 15.

Fig. 16.

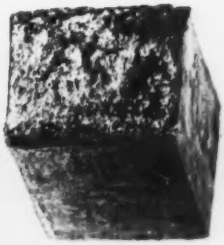
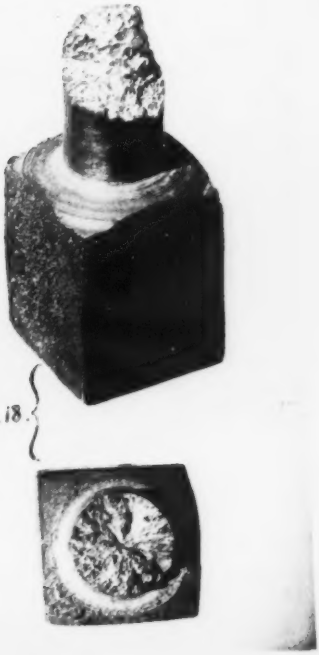


Fig. 17.

Fig. 18.



AUTOGRAPHIC ST

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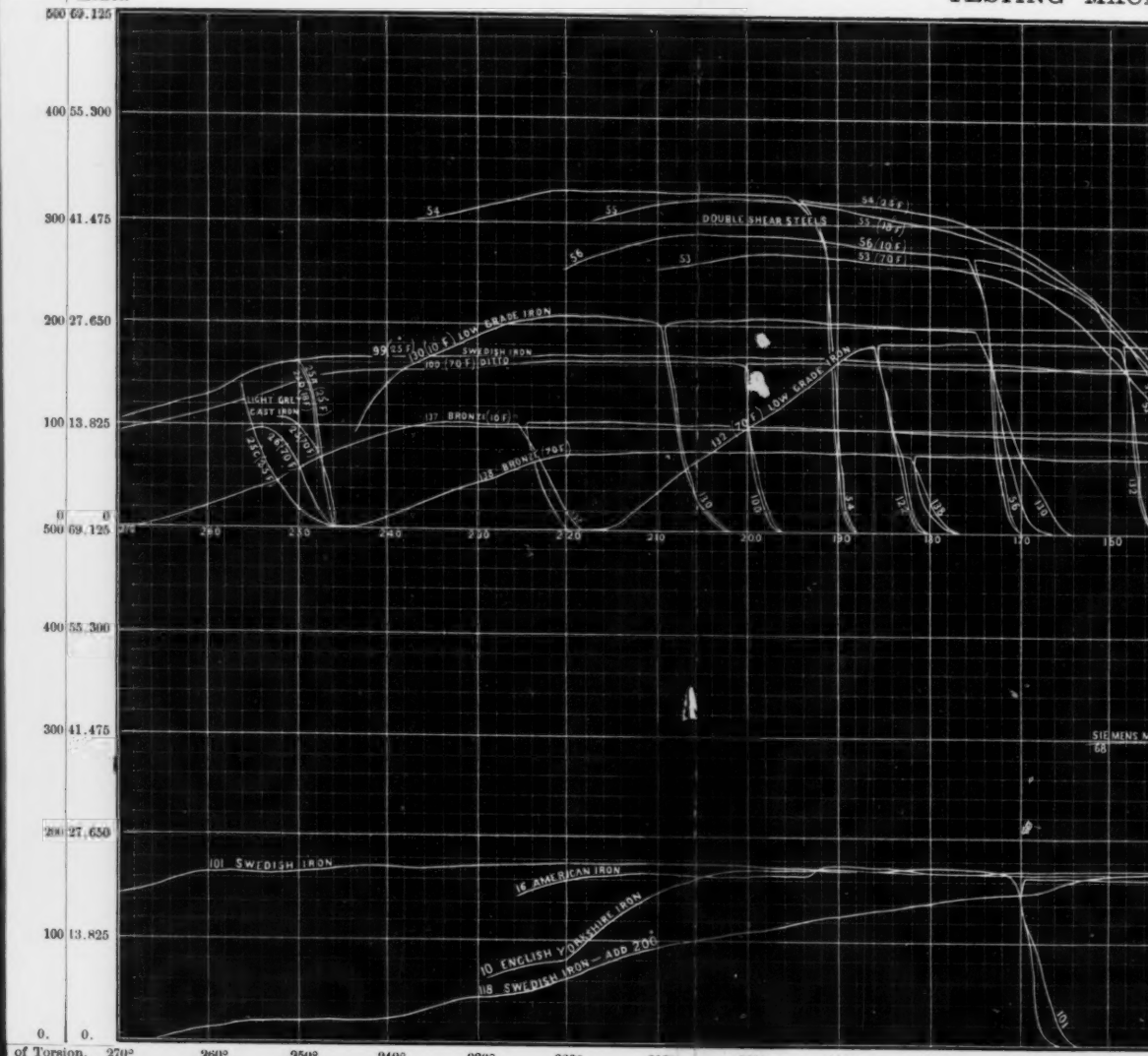


of Torsion.	270°	260°	250°	240°	230°	220°	210°	200°	190°	180°	170°	160°
Elongation.	.7800	.7360	.6910	.6460	.6060	.5640	.5190	.4800	.4400	.4010	.3635	.3275

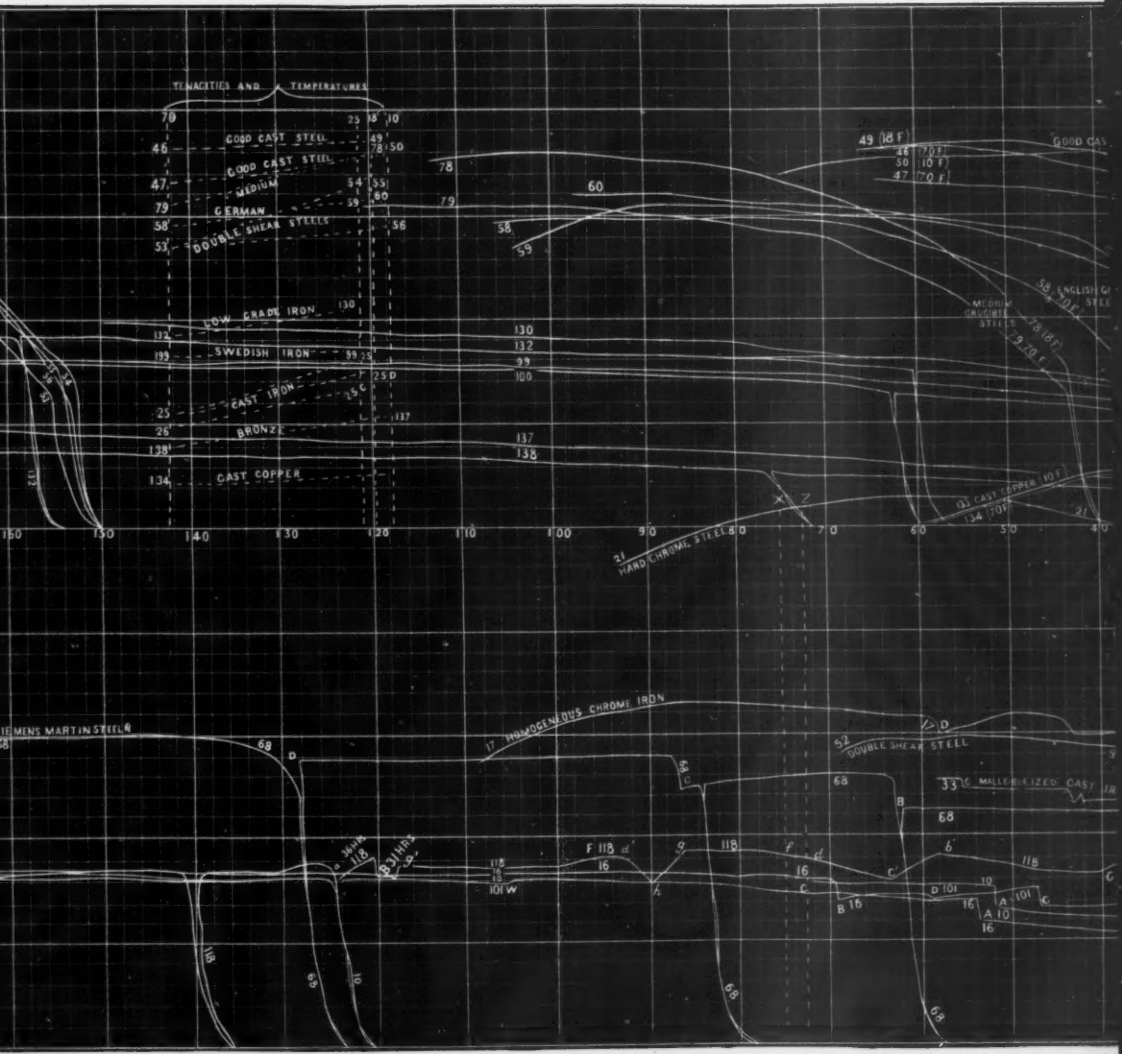
PLATE III.

STRAIN-DIAGRAMS OF METALS,

EFFECTS OF TIME AND OF TEMPERATURE UPON RUPTURE,

PRODUCED BY THE

ACHINE OF PROFESSOR R. H. THURSTON.



160°	150°	140°	130°	120°	110°	100°	90°	80°	70°	60°	50°	40°
.3275	.2920	.2585	.2255	.1955	.1665	.1385	.1135	.0910	.0705	.0530	.0365	.0285

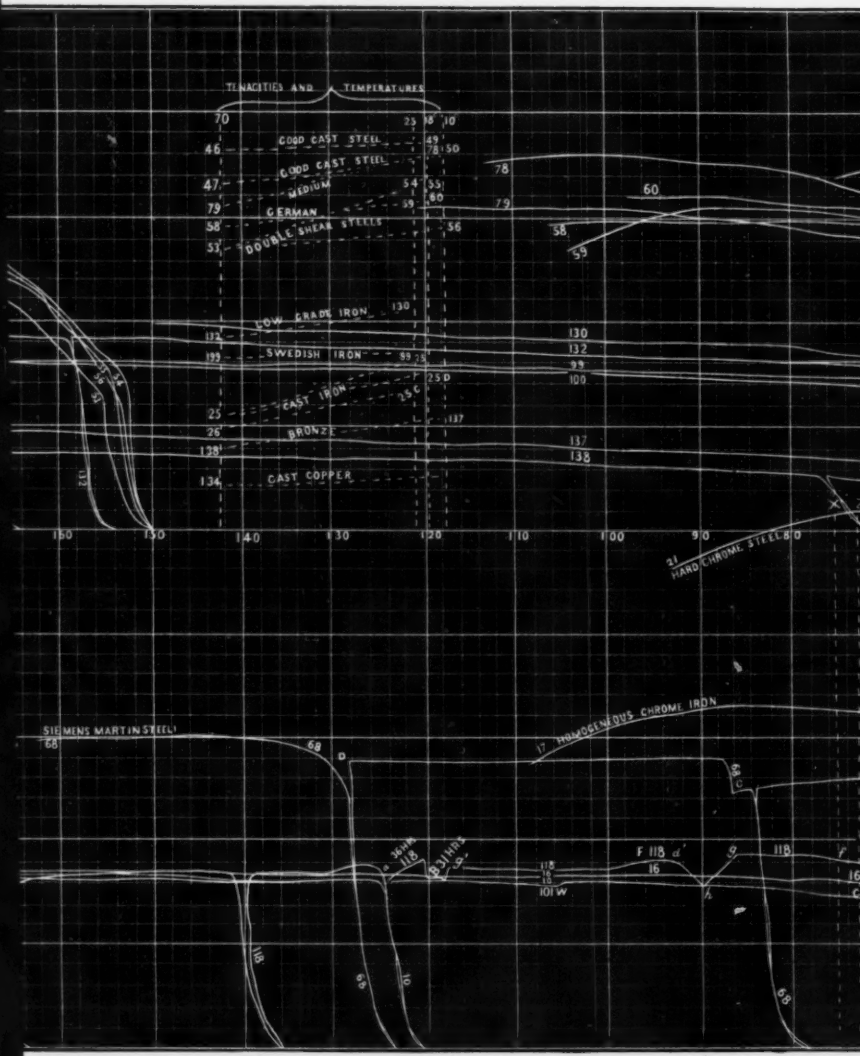
PLATE III.

STRAIN-DIAGRAMS OF META

THE EFFECTS OF TIME AND OF TEMPERATURE UPON RUPTURE,

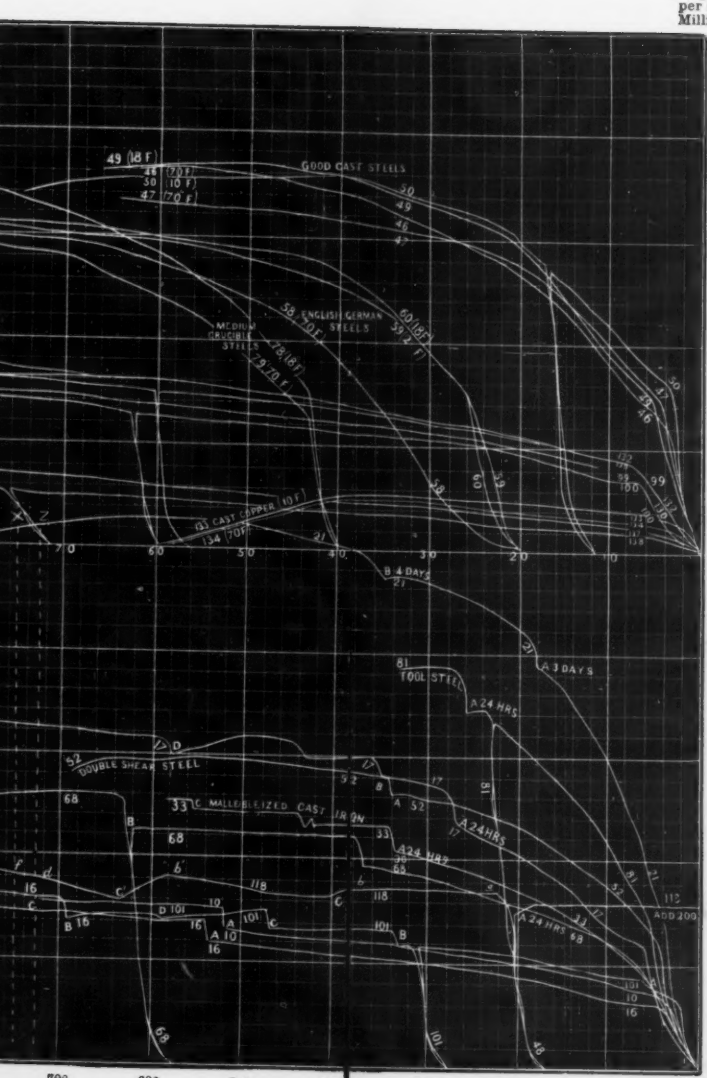
PRODUCED BY THE

MACHINE OF PROFESSOR R. H. THURSTON.



160°	150°	140°	130°	120°	110°	100°	90°	80°
.3275	.2920	.2585	.2255	.1955	.1665	.1395	.1135	.0910

TALS,



Approximate Tensile Resistance. Kilogrammes per square Millimetre.	Pounds per square Inch of Section.
105.460	150.000
84.369	120.000
63.277	90.000
42.185	60.000
21.092	30.000
0.	0.
105.460	150.000
84.369	120.000
63.277	90.000
42.185	60.000
21.092	30.000

70°	60°	50°	40°	30°	20°	10°	0°	Angle of Torsion.
.0705	.0520	.0365	.0285	.0130	.0060	.0015	0	Maximum Extension.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

LXXXII.

A RECORD OF EXPERIMENTS SHOWING THE CHARACTER AND POSITION OF NEUTRAL AXES, AS SEEN BY POLARIZED LIGHT.

A Paper by LOUIS NICKERSON, C. E., Member of the Society.

READ DECEMBER 18TH, 1872.

PRELIMINARY.—The experiments recorded in the following pages, were undertaken with a view to discover, whether the law of the "lamination of material in planes perpendicular to the direction of the pressure excited upon it," was a law incidental to a periodic* action of force, as pressure; or only consequent upon tangential stress, faults in the cohesion of material, or the running together of internal cells. The first of these was made by sending polarized light through plates of glass, compressed so as to be bi-refractive either in the character of beams or columns bearing weights.

As some exception may be taken to this apparently absurd means of obtaining the course of strains in materials used in construction, and because glass is itself a useless material for the sustaining of weights, it may be well that the experimenter's reasons for the proceeding should be given. The difference in effect between like action on two materials is not absolute, but only relative, therefore the effect of the action of force in glass must be similar to its effect upon other substances, excepting for amounts and confined qualities. One of these latter is transparency,

* The word periodicy being used to express a re-occurring phenomenon without reference to the time occupied.

but this becomes useful; and moreover, as the particular results which we wish to arrive at are derivable from certain general laws of the "doctrine of forces;" which must be precisely the same for all substances and only modified, not changed by any particular substance, and which the transparency of glass and its bi-refractive quality when strained enables us to watch with the eye of the body, in assistance to the eye of the mind; we may, in this partial case, completely ignore structural differences. So that, having become satisfied in regard to the general law, we may be better able to systematically admit the modifications. These modifications, indeed, would not appear to be very great. Glass, cast-iron and steel, are all amorphous bodies, yet all composed of segregated crystals, and during the experiments a connection was established in the characteristic form of breakage; all as a general result appearing to yield in this form: *Fig.* 1. when the fracture occurs in a beam between a supporting point and a point of pressure, and in this one when the fracture occurs immediately at a point of pressure: *Fig.* 7. it being readily noticed that the law of fracture is the same in both cases; Fig. 2 being similar to Fig. 1, with an opposite duplicature. Another and very important link of connection is believed by the writer to exist in the relative ratio of their respective resistances to tensile and compressive strains, as there is good reason to think that in this respect glass occupies a place between steel and cast-iron. These facts, however, become of less importance as we begin in the experiments to perceive the action of general laws, and to refer to their effects without regarding the vehicle which transmits them.

The form of apparatus used was the common one for experiments with polarized light, made without the microscope; namely, a bundle of glass plates—parts of window panes in this instance—6 by 8 inches. This was the polarizer, and one plate of heavy glass of the same size, blackened upon its back so as to reflect from only one surface was the analyzer. The first is set at an angle of 33° from the window through which the light is obtained, and the latter at an angle of 33° from the table upon which it is placed, or rather it is so placed that the rays of light reflected to it from the polarizer, come to the eye of the observer placed above at an angle of 57°. Between the polarizer and analyzer is held a small brass clamp used for compressing the object under examination. Looking into the analyzer, there is seen before compression, the clamp (a square frame of brass, with a thumb-screw moving from the top side to-

wards the bottom) containing, in position, according to the experiment intended, a slip of glass, or a piece of glass tube or rod.

The glass appears clear and ordinary looking, because of the ordinary light which is present, but if our apparatus were so closed up on all sides as only to admit of the polarized rays, *it would appear black, for in a neutral or unstrained state, it is not permeable by those rays.* Yet even now, by close examination, we perceive a cloudiness.

Beams.—Supposing that our object is a beam, as *G* (see Plate), and we tighten the screw a little, immediately we obtain clear spaces with curved outlines at the top and bottom of the slip, while the darkness of the middle portion seems to increase. This dark space, in all its changes of form, marks by its outlines the neutral or unstrained sections of the beam. As it at first covered the unstrained beam, so it ever, by well known optical laws, covers those portions of the beam remaining neutral or unstrained. Its increasing darkness, even, is only the effects of optical contrast; to pure polarized light it has always been black. Yet it does become more and more defined and distinct and narrower, as with the increasing pressure its outline becomes sharper. Soon, from the top and bottom of our beam, colored curves, each sharply distinct from the other, and having for their outlines the mathematical lines of strain, begin to make their appearance. As the pressure is more and more applied, they increase in number and brilliancy, moving always towards the neutral space. To a certain extent, there is a change of form, but the general curve is sustained until the period of fracture renders the image but a void of broken glass, the fractures of which may now be examined to obtain their position and character.

The images of *G* give a good idea of the results appearing in a beam, the ratio of whose depth to length is as 1 to 3.5, which rests upon two points of support, and which is pressed by a force partially local and partially distributed. The pictures opposite each other in the right and left-hand columns, are the positive and negative images of the same object in the same state.

The relative causes and properties of the positive and negative images, which together form each double picture, cannot easily be discussed until we have acquired, later in our experiments, a better knowledge of the character of the phenomena, after which, their great co-operative value will, doubtless, be admitted. They are, as arranged, images of the same state of the same object taken, first in one position, and then in another in the same plane, 45° distant, or by a revolution of the analyzer at each

90° distance. The words *positive* and *negative* are, of course, purely arbitrary, the negative image being the one occurring when the beam is horizontal and the column vertical.

The first matter which engaged attention and gave a definite course to the experiments, was the unsuspected and peculiar form and character of the neutral axis, and systematic trials were shortly commenced to arrive at a full understanding of its status, under all circumstances of load and position. It will be remembered by engineers, that during the long, patient and able researches of Eaton Hodgkinson, F. R. S., published in 1842, he took the ground that the neutral axis was shifted by increasing the weights used, and, ultimately, that its position should be found at a line which would equalize the sum of the stretching and shortening forces, by a corresponding distribution of material above and below it. He fixed this line for cast-iron—at the period of rupture—one-seventh the depth of the beam from the top, because the resistance to compression of that material was seven times as much as the resistance to tension. He occasionally found rectangular beams, which broke, so that the broken edges exhibited a crushing force for one-seventh of their depth, whilst the tearing forces occupied the other six-sevenths, thus apparently corroborating his theory by an exact distribution of strains, in accordance with his previous experiments upon the compressive and tensile resistances of the material. Therefore, and most naturally, too, his whole argument* makes it clear that he looked upon the neutral axis of the unloaded rectangular beam as a line at first coinciding with the longitudinal axis, but, which gradually rises towards the concave side at each accession of weight.

In 1855, Henry Barlow, F. R. S., made experiments upon cast-iron beams, purely to test this theory of Mr. Hodgkinson.† In the course of these experiments Mr. Barlow decides in direct contradiction to the

* Hodgkinson—Tredgold, page 384, Arts. 89 and 90, and pages 413-17.

† He says: "It has long been known that under the existing theory of beams, which recognizes only two elements of strength, namely, the resistance to direct compression and extension, the strength of a bar of cast-iron, subjected to transverse strains, cannot be reconciled with the results obtained from experiments on direct tension, if the neutral axis is in the centre of the beam. The experiments made both on the transverse and the direct tensile strength of the material have been so numerous and so carefully conducted as to admit of no doubt of their accuracy, and it results from these either that the neutral axis must be at or above the top of the beam, or there must be some other cause for the strength exhibited by the beam when subjected to transverse pressure.

"In entering upon this question, it became necessary to establish clearly the position of the neutral axis, and the following experiments were commenced with that object, but they have led to others, which are also described herein, and which established the existence of a third, and a very important element of strength in beams."

views of Mr. Hodgkinson, that the neutral axis in a rectangular beam under all circumstances lays through the centre of gravity of the beam sections, and that there is a third element of strength consisting of the lateral adhesion of the fibres—"something like friction"—which increases with the curvature of deflection.

Both experimenters start from the same point, viz.: that the transverse strength of a cast-iron beam is about 2.5 times as great as it would be to accord with experiments on its tensile and compressive strengths, and each records a different theory from the other in regard to the character and position of the neutral axis. This paper will show, that while both gentlemen were correct in the experiments made by them, these experiments were so diverse in character as to lead to two opposite theories by necessity, and by making a connection between the two sets of trials, also connect the observed phenomena and correct the theories. An attempt will be made to establish that the neutral axis is a flexible line, truly parallel to the top and bottom sides of the rectangular beam, and passing through the centres of gravity of its sections, only when the load is evenly distributed from end to end, or when the beam should be infinitely long; but that when there is a local pressure the neutral axis is more or less governed in its direction and form by the strain passing from the point of local pressure towards the points of support.

This mobile character of the neutral axis having been observed by the writer in his earlier trials, his first object when systematizing the further experiments was to obtain a law of direction, or rather to examine one already prevised. For this reason the negative image of column *A* was taken and recorded as evidently the simplest case that could be provided of direct compression. Then two slips of glass, *C*, were fastened in the press, so that when the pressure became heavy the two outside bottom corners should press against practically inflexible buttresses—the inside top corners press against each other—and the inside bottom corners and outside top corners of course be free. Thus there would be no tension in the beam, as a beam, and the compression could be governed by the screw until the coerced corners should be ground to powder. The image plainly represents two inclined columns like *A* each, or to speak more technically two "struts."

The simplest and most efficient way to conduct the experiment was obviously to make its different steps as closely consecutive as possible. The next object, therefore, should be to attain a very small tensile strain

with a great preponderance of compression. *D*, which was the next subject of trial, was a very short slip in proportion to its depth, with points of support very close together, so as to allow of but little flexure, and pressed like the rest by the small leather guard of the screw. Passing across from the negative to the positive images for reasons to be hereafter given, a dark line surrounding a blue figure may be observed in *D*, as in the positive of *C*. Now, this blue figure is the complimentary or positive appearance of the struts of negative *C* and *D*, and the dark surrounding line must, from the law mentioned in the first part of this paper, be neutral. In fact, the blue is in a state of semi-neutrality, shown by the small amount of the polarized light passed through it.

Confining the discussion to the neutral axis as it appears in the subsequent positive images, until experience shall have taught us the co-operative uses of both, let the beam which rests upon inclined but partially flexible supports be gradually lengthened in *E*. The blue is now less characteristically a strut, but the dark line still surrounds it. Only, however, on the side towards the middle of the beam is it plainly marked, and then not more so than in the new portion which forms a curve extending from one corner of the "pressure block" to the other. The neutrality in this case, then, is in the narrow band running from one abutment in a curved line to the corner of the pressure block, from thence in a curve to the other corner of the same block, and from thence in a third curved line to the other abutment, and in two branches proceeding from the abutments towards the upper corners of the beam; the space between the first and third and the branches remaining, as it were, semi-neutral. Looking backwards it will be perceived that the first and third lines, the branches and the semi-neutral spaces result from the strut strains, formed by the direct compression from the pressure block towards the abutments, and we may begin to believe that the second line has been formed by the introduction of a distinct tensile strain along the lower part of the beam, creating with a horizontal compressive stress the bending moment which is just breaking in to complicate the strains. Simultaneously with this bending moment it will be perceived that colored curves appear upon the top and bottom of the beam. This beam, however, is far advanced towards the point of rupture.

F is a longer slip in proportion to its depth, but the local pressure still upon its centre. In this, the general neutral space is a broken curve or

arch from one abutment to another, only sending off cloudy branches, towards the upper corners of the beam, and the lower corners of the pressure block. This slip has, as yet, been little pressed, and the yellow color of the curves shows that they have but recently made their appearance. In *G* is shown the image of a beam upon the border of breakage, generally similar in character to *F*, but the beam being longer in proportion to its depth, the arch becomes much flattened; we still observe the running up of the neutral space towards the same points, and further notice that they enclose the colored curves in so doing, like the quiescent water which lies between opposite vortices.

H a, *H b*, is a beam with the load equally distributed. The "strut force" has disappeared from the top and is only marked upon the beam, where, starting from the abutments, it becomes diffused about the centre, spreading, as might have been expected, when a local strain was changed to a general one throughout the whole top of the beam. Undirected by a local pressure, it becomes undefined and cloudy, the neutral space belonging to the regular bending movement of the beam, detaching itself and becoming a horizontal line for the first time. The colored curves are regular and nearly equal on top and bottom, and with the exception of the "strut force" which appears in the negative at the abutments, and the complimentary shading in the positive, there is no sign of local pressure. It may be noticed, too, how its colored curves are becoming more nearly similar in positive and negative images—how the colors are becoming more plainly complimentary in the two images and how an umbra, which is a true neutral axis, surrounds the colored curves of the negative and connects them across the centre. Looking back, we may now perceive this neutral axis, in direction nearly perpendicular to the one in the positive image, has followed us up from the column to this point and will, beyond.

I is a long beam in the ratio of its depth, and we see clearly how its length, or rather its flexibility draws the neutral axis out into a nearly straight direction, parallel with the top and bottom edges; yet leaving perforce a slight upward tendency in the middle of its length due to the local force.

J is different in character from all the foregoing, and in attempt, represents a railroad bar pressed upon at one point, resting upon two cross ties, and prevented from rising by others, to which it is supposed to be spiked. Again the self-same laws prevail, the neutral

space in general horizontal, tends to bend itself towards each pressing point, and as in all the other images to brush out when free.

Those beams of Mr. Hodgkinson's, by his own account, which plainly showed the neutral axis as being above the centre of gravity of the transverse sections of his beams, were *broken at the centre by a blow*. It was, therefore, as these drawings show, the local force and not the ratio of compressive and tensile resistances which governed its position. Mr. Barlow experimented with long beams, twice as long in proportion to the depth as any of those which appear here, and nearly 1.5 times as long as those mentioned by Mr. Hodgkinson in this connection. As the increase of the "bending movement," and the decrease of the "strut strain" keep pace with this lengthening of the beam in the ratio of its depth, he naturally found the neutral axis more nearly horizontal and less distorted than even in *I*. Beams of glass, with a ratio of 1 to 14, which is the proportion of those used by Mr. Barlow, show a very faint distortion from the horizontal, and measuring as he did from points one foot distant from each other, this slight distortion might easily have escaped the scrutiny of his micrometer. Indeed it must be almost indefinite in beams having a ratio of more than 1 to 10.

May not it then be said of the neutrality, as it appears by polarized light, that it is not a mere line in the mathematical sense of the word, but by reason of the cohesion of material or other cause, is extended to a breadth, and can never become a true line until the tensile and compressive forces become in relation to the cohesion—infinite; or, in other words, that although the neutrality of "stress" is a mathematical line, yet as in material it must require a certain definite "stress" to produce a "strain," the initial "strain," namely, that sufficient to overcome cohesion must be at some distance from the mathematical line of neutrality. Beyond this it appears that its longitudinal direction, like the directions of lines of strain, is not an arbitrary one, but is resultant from the relative qualities and quantities of all the forces in the beam, its evident place in physics being precisely that of the still water between opposing eddies or vortices.*

Positive and Negative Images.—It will be noticed that as yet no cause has been discussed for the difference between the right and left-

* The neutral character of the dark bands is most plainly shown by working at an open window, where a large quantity of ordinary light is reflected from the polarizer. The neutral portions are pervious to ordinary light; and by this means a white speck set upon the polarizer may be made to pass across them, by a motion of the head, from one set of curves to the other, as a star passes across a clear belt of sky from cloud to cloud.

hand views in the plate—the positive and negative images. To understand these, it is necessary to know, generally, that polarized light differs from ordinary light, insomuch that instead of vibrating from a centre outward in all directions, in the same transverse plane, the pulsations are confined to two directions which, when a completely polarized state is effected, are at right angles to each other. In this condition the pulsations in one direction are transmitted through certain substances, when the beam of light strikes them at particular angles, whilst the pulsations in the other are reflected, and *vice versa*. Secondly: if ordinary light, or either of these beams, are transmitted through a bi-refractive substance, there will be a division into two other beams at right angles to each other, and these falling upon an analyzing medium, properly arranged, will be either transmitted or reflected, according to the angle of their pulsations, so that the eye will be only able to discern the one reflected or the one transmitted. Thus the ordinary light coming through the window was polarized by falling upon the polarizer at an angle of 57°, one beam was transmitted, and, so far as we were concerned, wasted. The other reflected was made to pass through the object which, impervious to it in an unstrained state, became pervious and bi-refractive in a strained state, again dividing the one beam into two, at right angles to each other; the one of which is reflected to the eye from the surface of the analyzer, and the other transmitted through its face, but absorbed by the blackened back, until a revolution in part of the object changes the directions of pulsation and reverses the condition of the two rays.

When experiments were being made upon the first tubular bridges, it was found that a wave of "buckling" always followed an angle of 45°, from the top where the load was placed towards the bottom; even crossing the stiffners put in to retain this normal angle. This was similar in principle to what is here called a "strut strain." Evidently this would be impossible in equilibrium, unless there also existed a corresponding tensile strain at right angles. A resultant of these stresses in a beam would obviously be vertical. Again, it is known that the principal stress in an ordinary beam is the horizontal stress, composed of a compression in the top and a tension in the bottom, and that the resultant of these is parallel to them and is called generally the neutral axis. There are, therefore, two resultants at right angles to each other.

In the first section it was shown, that the true neutral axis of a beam was not necessarily the horizontal line, usually so called, but a line resultant from the actions of both the horizontal and strut stresses, which was

derived from the fact that the neutral or unstrained part of the beam coincided with this form. In a beam loaded with a distributed load, or one which is very long in proportion to its depth, the strut stresses assume angles of 45°, as before shown in the case of the tubular bridges, become individually small in comparison with the horizontal or flange stresses, and the horizontal stresses govern the resultant almost entirely, but as the beam is shortened, so that the strut or web stresses between the load and abutments, obtain an ascendancy over the flange stresses, then the strut stress interferes controllingly with the horizontal stress, and changes the direction of the resultant axis until in a column it becomes entirely vertical.

By a re-examination of the drawings it will appear that when the strut strains are in an ascendancy over the horizontal strains, the negative images are by far the brightest, and although they afterwards become duller—as the strut strains become relatively less—yet the preponderance of vertical stress is always evident in them On the other hand, the instant a tensile stress was introduced, so as to oppose the compression or strut stress, and resolve the effect into horizontal or "flange" stress, the positive images became brighter, and continued so to grow *ceteris paribus*, with the increase of the beam in length, and consequent relative increase of tensile and flange strains. Further, it should be remembered, that divers stresses may exist in the same piece of material, crossing each other, and probably being mutually neutralized at the crossing, but continuing on afterwards as individual and intact as when first they met.

As the glass is pervious to polarized light, when strained, and impervious when unstrained, we can only regard the light parts as strained in proportion to their brightness, and the dark portions as unstrained or neutral; and as the strain is the cause of the bi-refraction, it follows that the latter must be governed by the laws which govern the former. Further, the resultant angles of the strains are rectangles, as are the angles of polarization, and as the strains change in character, so do the polar images change in correspondence. It follows, then, that the images are pictures of the strains.

But this may be examined in another manner. If all the light—except that which falls upon the polarizer—were dispensed with, the object seen by the eye without the intervention of the analyzer, would yield no appearance of color, but only white light. Now, all the stresses in a *beam*, when united, must be in equilibrium, hence the light which measures and was produced by them, should also be in equilibrium, or in other words,

colorless when the two rays are united. From this is deduced that the positive and negative images are complementary in form and color.

Further, it may be fairly assumed, should these reasonings be correct, that if there is any one point in the object which is under no strain, whatsoever stresses be applied to the beam, that point must remain dark under all revolutions of the object or analyzer. Now, there is such a point in the centre of the beam, nearly in the middle of the primary neutral axis, and between the forks of the strut strains, and which, by laws already discussed, resolves itself into a space, and this dark space is always observable in the analyzer, forming a sort of nucleus, from which the changes from positive to negative and the reverse take place during the revolutions. It is obvious that this point marks the crossing of the resultant axes, the one formed by the "web" or strut strains, the other by the "flange" or horizontal strains, as before discussed.*

Colored Curves and Secondary Neutral Axes.—The appearance of the light around the neutral band is the first symptom of distortion in the positive image, but soon there appears upon the top and bottom edges faint segments of yellow. At the lower side this occupies a large portion of the distance between the points of support, but at the top it varies with the arrangement of the load, the chord to the segment having nearly the length of that part of the beam directly pressed by the load. If the load be uniformly distributed over the whole length of the beam, the segment is similar at top and bottom, but if this be local, say consisting of a block of hard sole leather as a guard to the screw, then the segment will have about the same length as the block, and two smaller segments, differing a little in form, will appear at the sides. Beyond these again bright curves appear, as though preparing for the formation of more colored segments; but the writer has never succeeded in bringing out more than three, with sufficient distinctness in the colors of the extra ones to be satisfied as to their character. It may not be amiss to remark here, that all the images and colors appearing were quite as distinct as the shades represented in the drawings.

As the pressure is increased, the yellow from both edges moves forward towards the neutral band, becoming soon a zone, and a segment of red succeeds. Both these continue to move forward with the increase until the red, too, becomes a zone and a segment of blue appears. This is the

* The writer will endeavor, at a future time to present the concordance between the mechanical and optical phenomena, more in detail and with greater precision. For the present, relying mostly upon the success of the ultimate experiments to prove the law conceived to be true, he has preferred rather to suggest the line of argument than to closely follow it.

end of the first series, and we mark with interest that with the blue or between that and the red there came a dark mysterious line. Turning the screw continuously we force the yellow, red and blue to move forward, the blue becoming a zone, and bearing its dark, line with it, which constantly grows darker, and yellow, red and blue again chase each other upon the field to form a second series. The second blue also has its dark line. So far I have found no glass to stand more pressure than this.

Let us now remember Mr. Barlow's words: "But they have led to "others (experiments) which are also described herein, and which "establish the existence of a third and very important element of "strength in beams."

In loading a beam, it is probable that the pressing force first seeks to connect itself with, or as it were feels for the points of support; afterward the bending action commences. Now, suppose the load to be very small at first, and gradually to increase, there must be a time, for the new resistance can only exist, according to Mr. Barlow's experiments, after the beam is bent somewhat—when this resistance is only just equal to that portion of the moment of the compressive force (we are discussing, for simplicity the top part of the beam), which is directly opposed to it, and which may therefore be called the bending stress. Taking the vertical section through the middle of the beam for the discussion, we may fairly suppose that this occurs just when the dark line between the first blue and red comes upon the glass. Mr. Barlow has shown that the new resistance or resistances increase as the beam is bent, but as we know that by the increase of load the beam is still further bent, we derive that the bending force increases faster. Therefore, if at first the bending resistance, multiplied by its minimum statical arm, was equal to the bending stress, multiplied by the distance from the neutral axis to the top of the beam as its maximum statical arm, which at equilibrium it must have been, it must at last be equal to the increased bending stress, multiplied by a shorter arm, itself being now multiplied by an increased length of arm, whose axis is at the top of the beam. The line of equal moment then between these two forces must have moved down in the interim nearer to the primary neutral axis, and must continue to move downwards towards this axis as the load is increased. This is exactly the action of the dark line in the blue, and its appearance, to one frequently engaged in examin-

ations by polarized light, is that of a line of equal antagonistic forces—in other words, a neutral line.

In a first examination of this matter there seems to be something anomalous in the fact of a neutral axis existing amidst distorting forces, of a line in the top part of a beam which is neutral, yet growing shorter or in the bottom part and retain its neutrality whilst increasing in length But it is not so; for if we should load a beam to bending, and then without changing the relative status of its flange strains compress or stretch it bodily in length, we should have precisely the same phenomenon—a line which is a neutral line between two forces—yet increasing or decreasing in length by the operation of other forces acting independently of the first. There might be a modification of form besides the change in length, but there could be no cause for annihilation.

Further notice of the movement of this secondary neutral line (if we may call it so) towards the primary neutral line, suggests that should the two come together, then, according to our former reasoning, the bending stress, multiplied by its smallest statical arm, would have become just equal to the bending resistance, multiplied by its longest possible statical arm (from the top of the beam to the neutral axis), and that any additional increase of the bending stress, however small, would bend the beam infinitely, because totally unopposed; or in other words, the beam would break. This reasoning was curiously confirmed in the following manner; whilst watching the consecutive phenomena occurring in several slips, set one upon another, thus forming a laminated beam, with a small hand microscope, all the slips, save one, suddenly broke. For some reason, perhaps from the elasticity of the broken glass, or perhaps the screw which had been shortened by the reaction of all the slips, began to recover its length when only resisted by one, the secondary axis continued to move forward, so that the observer, not having the screw to manage, could watch its movements at leisure. As it neared the primary axis its curve began to take the form of a bow, the two quarters passing the middle in the direction of motion, and when very close two loops were prominently advanced, first one and then the other. As the first touched the primary axis a sharp "click" was heard, and fine fragments of glass flew like hail from the sudden rupture, which followed just at *that point*. Immediately the second loop touched with precisely the same phenomena, when the vision was gone and only fragments of glass remained. It is to be regretted that the flying of the glass particles made it impossible to watch the movements of the secondary axis in the

lower part of the beam, or either of the dark lines in the second blue, but closer preparation at another time, when the phenomena is looked for, may lead to interesting results.

COLUMNS.—The column appeared to us at first as incidental, taking a subordinate place in discussing the general laws of a compressed beam. We shall now see that its own phenomena are well worthy of deep study —that it resists pressure by strange geometric laws, and marshals its forces to oppose coercion from a vast depth of mathematical resource.

Although with glass, probably on account of its hardness in relation to the screw, it seemed impossible, before breakage, to obtain more than from 3 to 6 rings at either end, yet when a softer material, such as copal, was used, the column was wreathed in colored bands from top to bottom. Observe the dark band between the blue and red. Its position is invariable whenever these two colors come together, and from the first my suspicions were decided that it marked either an unstrained path in the column or a division of equal antagonistic forces. In fact, we must not regard the bands as horizontal strata, but rather as sections of oblique planes which follow the directions of stress. (See diametric sections *A* and *B*, which are the negative and positive images of the same object.)

It was with a view of assuring myself of the truth of these suspicions that I one day asked permission to search through the store-room of the Illinois & St. Louis Bridge Company for corroborative phenomena. In this room were many objects, crippled and otherwise, which had been tested in the testing machine belonging to that company, and I was in hopes of finding in changes of form some assimilation to the arrangement of colors in the glass columns, and especially something to corroborate the neutral character of the dark line, as already provided. There was no want of testimony in any piece of the existence of a wave action in the pressure, but in the solid bars this was confined to a cone like flattening (upsetting) at either end, and a determined "bulging" in the middle for those compressed, which was reversed in those subjected to tension. It was common to steel, iron, wood and cement cylinders (see *L*). Attention, however, was soon concentrated upon the tubes, all of steel, which plainly exhibited a periodic wave apparently at regular distances from end to end. Not being prepared at the time to measure them carefully they were left for a future occasion.

Still further to test the matter upon material, which would yield form instead of color to the forces brought to bear upon it, arrangements

were made by me for a change of experiment. Definite trials were made upon brass tubes .45 inches outside diameter, .35 inches inside diameter, and a length necessarily confined to about 1.5 inches by the small clamp used. These were compressed longitudinally by a screw until the form assumed was permanent. *M* gives an idea of their original and assumed appearance. These trials having been carried through some 12 or 15 tubes and the results subjected to measurement, the store-room of the Bridge Company was again visited, and a scale used to measure, as correctly as possible, the distance between the waves on the steel tubes. Then, with very little labor, the results were tabulated, and with not much more (since either the outside or inside diameter, or the radius of gryration, were almost obviously the base, and some oblique line proceeding from the axis of the tube outwards, as certainly the hypothenuse of a triangle of pressure) the following empyric expression was derived, which, so far as I can discover, will locate any wave in relation to another, the material of the tube being of course equally resistant from end to end. Measuring from ventral to ventral, we have when x = distance, R = outside radius, r = inside radius, a = constant quantity varying with the material alone and dependent upon the angle at which it most easily yields.

$$x = a \sqrt{\frac{R^2 + r^2}{2}}.$$

This formulæ seems for brass tubes to become, as nearly as could be discovered,

$$x = \tfrac{5}{6} \sqrt{\frac{R^2 + r^2}{2}},$$

and for steel tubes,

$$x = \tfrac{7}{8} \sqrt{\frac{R^2 + r^2}{2}},$$

a being equal to unity. The steel tubes measured, varied from 1.5 to 2 and 5 inches in outside diameter, the waves on the only one of the latter present being perceptible, but faint except at the ends, from 1 to 42 inches in length, and from .05 to .25 inches in thickness of wall. They were 9 in number.

It became now necessary to show some cause for these phenomenæ and to bring their relations with the colored rings in the glass column, and especially with the dark line between the blue and red, into closer connection. With this idea a theory already previsedin part from the colored rings was taken up, with what appears to the writer a successful issue, as follows.

We learn from Rankin's "Civil Engineering and Applied Mechanics," "that no tangential stress can exist as a simple stress, but must be ac-"companied by another stress, upon another plane, and that these two "stresses have a resultant, which becomes a maximum when they are "equal, departing from each at an angle of 45°."

Let us then consider the condition of a solid or hollow column, pressed evenly over the top by a great load, and resting upon a flat surface at the bottom. Evidently the weight must exert a tangential stress upon the walls of the hollow column, or the edges of the ultimate particles of a solid one, and as the longitudinal section of the column is rectangular, we may look for the accompanying stress to be at right angles to it. Let us for convenience consider these stresses to be equal, their resultant will have a direction between them and 45° distant, and if *K* represent the column, *A A'* and *A A''* will represent the resultants of the stresses. But these resultants act to push the material outwards at *A'* and *A''*, and as their action is, therefore, tangential or distorting, there results from our first law, and directly from this effect, two other "accompanying" stresses, *C, A'* and *C, A''*, which tend to draw the material atoms from *C* towards *A'* and *A''*. Now, where these stresses interfere at *A' A''*, they mutually neutralize each the vertical component of the other, so that there only remains at *A' A''* horizontal forces, and where they interfere at *C*, they mutually neutralize, each, the horizontal component of the other, so that there only remains at *C* the vertical component, equal to the weight at *A*, with which we commenced this discussion, and the plane *B, B*, is precisely in like condition as regards stress to the plane at *A*. It follows as a corollary that like conditions below *B, B*, will produce like results to those at *A*, and *B, B*, and so on for any number of equal divisions or periods, and it also follows that such planes as *A* and *B, B*, &c., are unstrained by horizontal forces, and should, therefore, appear dark in the positive image, and that such planes as *A' A''*, &c., are unstrained by the vertical forces, and should, therefore, be dark in the negative image, which, in the bi-refractive glass columns, under polarized light, we find to be true, whilst in the metal tubes compressed we find a concording form.

L shows the form of the wave assumed by thin brass tubes—probably any thin tube—which has for its periods the same formula as the thicker ones, but differs in the vertical fold, and the consequent interlaced triangular section. It is probable that the vertical fold does not occur from any definite change, but only from a modification of our law,

and may rather be regarded as a buttress, thrown out by tubes of a certain thinness, to resist transverse bending, and from thence propagated, by the natural pulling of the force of its creation, upon straight lines, and located by the well known properties of the equilateral triangle as a perfect figure of equilibrium. Its position in the succeeding period midway between those formed above it, so that one triangle interlaces with and bisects the other, may also be accounted for by the fact that the middle of the side of the triangle is its weakest point, and the new buttresses are thrown out there in consequence of greater yielding at that place.

Remarks on Attendant Phenomenæ.—It was apparent during the course of these experiments on columns that each piece of material was mechanically polar, or, in other words, had a favorite end. This was shown in glass and brass by the end against which the screw-plate worked, yielding first. In the thick tubes, and in transparent columns, this was followed by the other end, and in the same tubes, and in softer transparent material than glass, by the intermediate ventral points. Of course, after a certain yielding in glass columns, the balance of compression was against the brass screws, which then yielded more in this direction than the glass, the glass column ultimately giving way by splitting down perpendicularly, as all vitrious substances naturally do, except a transverse crushing where the rings had appeared, making a nearly cubical fracture. In thin tubes the waves formed from the screw or active end towards the other end consecutively.

When, as frequently occurred, the tubes were not of exact length to make a regular succession of equal waves, then the difference was invariably made upon the base or passive end by a half wave longer or shorter than the normal half wave shown at each end in the drawings. In only one case was an abnormal wave noticed, this in a brass tube with an evident soft place in the material, which divided into five waves in the usual space for four, one wave coming within the soft place.

In the proposition it will be noticed that the lines $A\,A'$, $A\,A''$, $C'\,A'$ and $C'\,A''$ are straight. This was only the natural result of an attempt to investigate forces proceeding from one point to another. Subsequent experiments, which have not been as yet sufficiently wrought to publish, lead me to regard these as curved lines, and to consider the figure $A\,A'$, $C'\,A''$ as approaching an ellipse corresponding to the rhombus shown. The figures in the lower divisions would then, of course, suffer the same change from the same law.

I have thought if large, hollow columns yielded under the same law, and it should appear, as apparent from the proposition and from other facts, that the law governs before the limit of elasticity was reached, then by reinforcing columns by bands shrunk on at the places pointed out by formulæ, we might greatly strengthen them at a small expense for additional material in the shape of rings (See *O*).

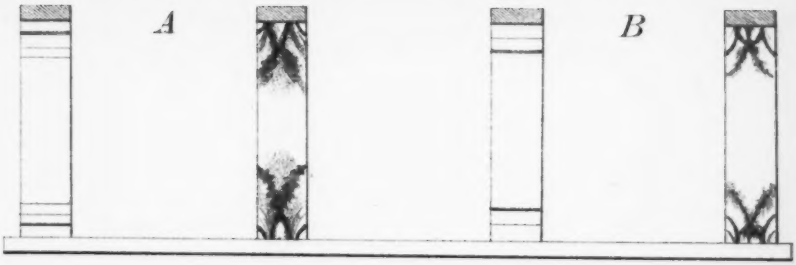
In conclusion, it seems to me, subject to higher decision, that the law of the periodicity of force in pressures, and probably in tension, has been fully proved in this paper, which will assist in accounting for the effects of re-occurrent applications of force, and for the law of lamination under pressure, in planes perpendicular to the direction of pressure.

Mr. Charles Macdonald.*—The experiments upon which formulæ in common use, for the proportion and strength of beams, are based, were made upon cubes of the materials tested; and it is found that the constants derived are inapplicable to ordinary cases in practice. It is known that a beam, loaded so that its outer fibres by formula should be under a maximum strain, will stand a greater load. The constants employed in calculating the strength of a beam should be determined from experiments upon a similar beam of the same material, and not those given for direct tension and compression.

Mr. Alfred P. Boller.—A recent comparison of various standard formulæ for the strength of beams, made by taking a given beam and calculating, with each formula, its safe load, gave us follows:—two, 25 tons; one, 30 tons; and one, 37 tons. The last was from Mr. Baker's formula, in which he has introduced the resistance due to flexion. The beam taken was 3 by 6 inches, and 5 feet long. Engineers should know more of the strength of materials. The formulæ used involve the application of constants ranging variously from maximum to minimum strength. These often differ widely and, therefore, give uncertain results.

Prof. De Volson Wood.—The point of greatest interest to me in the paper read, is whether the neutral axis is a line, or a space as water in an eddy. In regard to the neutral axis and strength of beams, as determined by ordinary rules, the fact is, if we assume in a bent beam that the extensions and compressions are proportional to the distance of the elements from the neutral axis, and take the modulus of tenacity or of crushing, as the case may be, for the modulus of rupture of the beam, a beam proportioned in accordance therewith will have an excess of strength.

* A portion of the following was presented, through the Secretary, February 19th, 1873.



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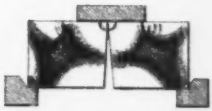

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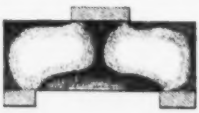

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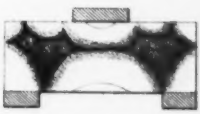
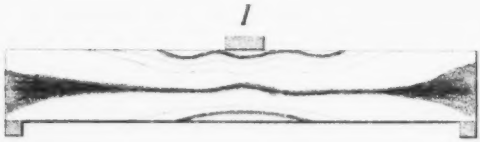


G

Ha
Hb
a
N
b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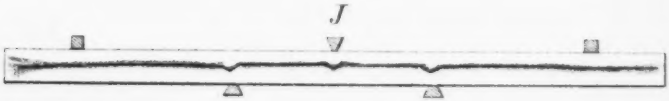
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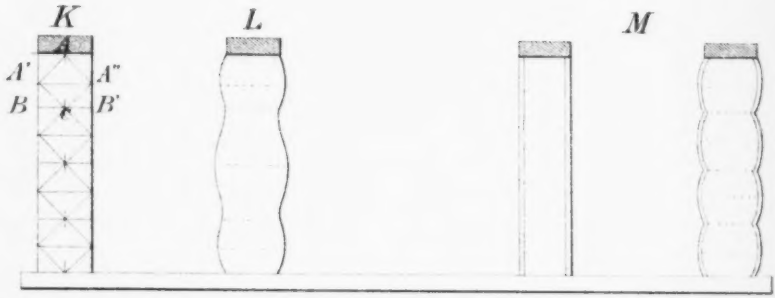
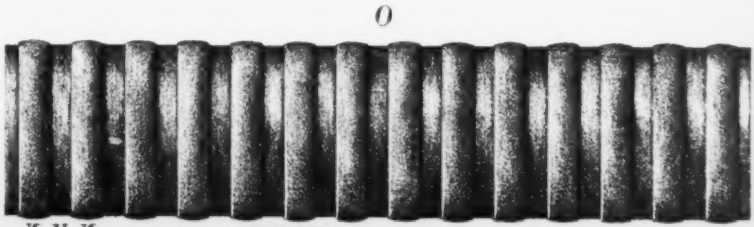

K
L
M
A'
A''
B
B'


O
x-y-x
y-d √(R²+r²)/2

To illustrate this, take the case of cast-iron, which is the best example in practice:—the mean tensile strength (T) of cast-iron is 16,000 pounds, and the mean crushing strength (C) is 100,000 pounds, or, for convenience, six times as great as the other—96,000 pounds. If the beam is rectangular, supported at its ends and loaded in the middle, we have the well known formula:—

$$\tfrac{1}{4} P l = \tfrac{1}{6} R b d^2$$

in which P is the weight, l the length of the beam, b its breadth, d its depth, and R the modulus of rupture. According to the theory, the value of R should be 16,000 pounds, but according to experiments it is found that the mean value of R for cast-iron rectangular beams is about 36,000 pounds. Former writers do not explain this discrepancy, but simply admit a defect in the theory, and pass it over by saying that a proper coefficient of safety will make the formula a safe one. Barlow detected a new element of strength, which he called "resistance to flexure," a term which is unfortunate, as all the forces in a beam which resist bending, are resistances to flexure; "longitudinal shearing resistance" is a better term, and one now used. This force is the resistance of the fibres to being drawn over each other. If there were no such resistance the fibres would retain their original length. To illustrate, suppose that several very thin but perfectly smooth boards of equal lengths were placed above each other, and supported at the ends. If now the pile of boards is bent, all the boards will retain their original length, and the ends will set past each other, as shown in Fig. 3, but the general range of the ends will be parallel to each other. If we suppose that there is friction between the surfaces of the successive boards, the upper ones will necessarily be shortened, and the lower ones elongated. In a beam, the consecutive elements are united by cohesion, and Barlow found by a critical examination of those of cast and wrought-iron, that the transverse sections remained normal to the neutral axis during flexure, and in this case the ends will not be parallel, as seen in Fig. 4. This being so, and bearing in mind the analogy between this case and that of the thin boards, we see that besides the forces which produce direct elongation and compression, there is a force between the consecutive elements somewhat analogous to friction:—this is longitudinal shearing resistance. Barlow found the laws which govern this resist-

Fig. 3.

Fig. 4.

ance at the point of rupture, and modified the formulæ accordingly. The mode of his investigation and his results are fully described in the "Civil Engineer and Architects Journal," Vols. XIX and XXI. I consider them the most weighty and valuable sets of experiments that have ever been made upon beams, so far as theory is concerned.

It is evident, the ordinary law that the strains vary as their distance from the neutral axis cannot apply to I sections, for to secure such action there must be at least a continuity between all the elements of the flange and those at the neutral axis. Barlow made his formulæ applicable to this case, but it is equally evident that they cannot be rigidly correct. In all flange sections there is a peculiar combination of the strains about the angles where the flanges join the web, and it seems impossible to assign the laws which govern them.

Prof. Norton, of Yale College, determined from direct experiment, the fact of a transverse shearing resistance in beams, and also the laws which govern it for rectangular beams. His article was published in the "Journal of Arts and Science" and in "Van Nostrand's Eclectic Engineering Magazine." I had detected the same analytically. If transverse elasticity only existed in a beam it would be deflected. To show this by a figure,—suppose that short portions of the beam were perfectly non-elastic, and that between these were elastic sections. Then will a beam which is supported at its ends, and loaded at the middle be deflected, as in Fig. 5. If the beam has a continuous transverse elasticity, it becomes evident that the deflection will be as shown in Fig. 6. The deflection due to this cause is additional to that due to Navier's theory, while longitudinal elastic shearing resistance will diminish it.

Fig. 5.

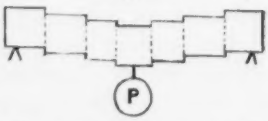


Fig. 6.

It is worthy of note that the strength of beams, as determined from Barlow's new formulæ and constants, agrees remarkably well with the actual strength for all the forms which he used. It is hardly to be expected that a simple law, or laws, can be given which will be applicable to all the forms of beams now used in practice. The constants which are determined from solid rectangular beams will doubtless apply practically to solid beams of all dimensions, but not to other forms. For instance, if solid cast-iron beams give $R = 36{,}000$ pounds, it will not be safe to use this value for hollow beams, or I beams. For such beams $7 = 16{,}000$ pounds is safe, but may give an excess of strength.

Mr. Boller.—It is best to make a beam proportioned similar to those to be used. Break it and deduce therefrom a constant. The formulæ for I beams are safer for thin than thick webs—say ½-inch web and 4-inch flange.

Prof. Wood.—The experiments of Baron Von Weber, "Engineering," 1870, showed that the web had never been made too thin. Where the flange and web are joined there should be a large curve.

Mr. Macdonald.—In T. C. Clarke's description of the Quincy Bridge, there is a statement that 12-inch I floor beams, 15½ feet long, suspended in pairs from panel points about 12 feet apart, and carrying a single track of 4 feet 8½ inches gauge, scarcely deflected under a maximum engine load, which caused a strain upon the outer fibres at least equal to 14,000 pounds per square inch, according to formulæ in general use.

Prof. Wood.—We should not proportion structures in reference to the ultimate strength of the parts, but in reference to their elastic limits. The elasticity of the material should not be damaged. Some irons which are very tenacious have a low limit of elasticity, while others, and especially certain grades of steel, seem to preserve their elasticity for a strain nearly equal to half their tensible strength. Capt. Eads, in the construction of the St. Louis bridge, insists upon preserving the elasticity of the steel, and very careful experiments are made to determine its elastic limit.

Col. W. E. Merrill.—I regret I cannot give this as thorough an examination as I could wish. It opens a field for investigation that promises the most useful results in a matter of vital interest to Engineers and Architects, and in fact to every one: the laws of the action of strains on materials used in construction, and the proper method of meeting and sustaining them with assured safety. Being quite unfamiliar with the use and laws of polarized light, I do not feel competent to discuss the very interesting results obtained, but the method chosen, and the means pursued, seem most admirable. The plan of using glass to find out the laws of steel and iron, reminds me of the practice of physicians in ancient times of determining the laws of the human body by experiments upon inferior animals. The limitations of sense having apparently put an insuperable barrier to the examination of the interiors of opaque bodies under strains, the method by analogy seems the sole one that is open to us, aided of course by what we can see on the surface, and detect after fracture.

I have often thought it desirable to be able to see exactly how the strains in the different members of a truss bridge were working during different positions of the moving load. The method of Mr. Nickerson

seems to offer that long-needed means. Further experiments will, probably, enable the experimenters to decide with fair accuracy upon the magnitude of an unknown strain on a glass column, by comparison with the observed effects of a known strain upon a similar column. I would suggest that a model of a Howe truss bridge be made with glass members and metallic joints, so constructed as to permit the ready insertion or removal of glass columns, acting as struts, ties, and chord segments. I believe that such a model can be made and so adjusted that broken parts can easily be replaced. If then the method with polarized light be used, valuable information about points now somewhat obscure, could be obtained so as to command universal credence.

The proposed strengthening of tubes by external rings, at regular intervals, is a very curious result of the experiments, and if confirmed by practice, will be a valuable discovery. The whole discussion illustrates most admirably the inter-dependence of all branches of physical science, and the absurdity of considering any discovery useless, no matter how far it is apparently removed from a possibility of practical application in the useful arts. Hitherto the polarization of light has seemed a curious and occult phenomenon, of no particular utility, and only interesting to the man of abstract science, and here it suddenly springs up as a trustworthy means of unveiling the secrets of nature, and promises as great a future of discovery as the spectroscope.

Hon. William J. McAlpine.—As a practical question, the Civil Engineer has almost never to deal with glass as a material of strength, but it must be evident to all, that it will be affected in the same way as any other material of similar construction, and hence that the effects of strains upon glass will be similar to those upon cast iron. At least the experiments made with such apparent care would be of great value in determing the law upon the subject.

It is a singular characteristic of this age that every discovery in science is made applicable to the service of our profession. Five years ago Prof. Airey (the Astronomer Royal) gave a paper on a new system of calculating the strains on each member of a truss, by a model in steel, which he had exactly duplicated. When the model bridge was loaded in all of the various ways, each member was struck, and the corresponding duplicate was loaded gradually until the two were in accord. The load on the duplicate indicated the strain on the corresponding member of the truss in position and under its load or strain. There we had *sound* to determine a most important engineering question; and now we have *light* in one of its forms of action.

GEN. J. G. BARNARD.—I have been unable to give the matters presented by Mr. Nickerson the study required for a full appreciation. I would remark, however, that the peculiar circumstances under which the experiments for the neutral line or axis are made, are really foreign to the subject of the position of that axis as it is presented to the engineer in connection with the theory of the strength and stress of beams. That theory and the formulæ deduced, are, like nearly all our mechanical theories of the properties of matter, based on somewhat rude assumptions. Moreover, the formulæ for "beams" really suppose the length to be very great, compared with the depth—practically infinite. No one supposes that, in pieces so short as the glass specimens experimented upon, the "strains," "stresses" and "strength" would be represented by the usual formulæ for "beams." That theory ignores all "strains," in the technical sense of the term, except those due to the elongation of fibre on one side of the neutral axis, and its forced contraction on the other. It is far from being a true exposition of the phenomena which are developed by the bending of a beam. The truth is limited by the imperfection of elasticity, and by any amount of deflection given to the beam beyond a very limited and exceedingly small amount; for tangential (or shearing) strains are disregarded.

The experiments upon pieces so short as those used by Mr. Nickerson develope phenomena which scarcely belong to the theory of beams, or to the position of the "neutral axis" in the usual acceptation of the term. I do not, in saying this, depreciate the value of experiments of that kind. They are directly applicable to the illustration of the strains produced in columns—and in all pieces subject to similar stresses. I would remark, too, that glass, to the extent to which it is capable of receiving change of form without rupture, is almost perfectly elastic. Iron is far from being so, at least beyond the narrow limit to which it may be distorted without "set."

I am not familiar with the investigations of Mr. Barlow, but if the stress in a bent cast-iron beam be a "uniformly varying" one, he must prove, I think, that the co-efficient of elasticity is constant for all degrees of extension. That co-efficient for wrought iron varies in a table given by Morin (taken from Hodgkinson) to an extraordinary degree; being for extreme extensions, only about $\frac{1}{2}$ of its value for very small extensions; though within limits of practical strains—say to about 18,000 pounds per square inch, the variation is small, say $\frac{1}{12}$ of its maximum. For cast-iron the co-efficient for extension exhibits some variation also, in practical limits of stress, that is, within limits of safety. For compression, both of

cast and wrought iron, the co-efficient is (within practical limits say up to 15,000 or 18,000 pounds per square inch) more nearly constant in both materials.

In the experiments* made by me on the strain and rupture of beams of forged and cast-iron (of dimensions never before experimented upon, I believe), the co-efficient of elasticity as deduced, from the assumption of a "uniform varying stress" in the cross section, and a central neutral axis, varies in one of the forged specimens from 1,482,000 to 9,558,600 pounds per square inch—the smaller co-efficient corresponding to the extraordinary extension of fibre of .0463 and a "tensile strength" of 68,450 pounds per square inch—(calculated from the amount of deflection of the beam)—and this extraordinary extension and this high tensile strength is exhibited in the iron of a *heavy forging*—i.e. in a forged beam, 15 by 12 inches in rectangular section. On the other hand, a cast iron beam of the same dimensions, rupturing under a force of 276 tons with a flexure of ¾-inch (the supports being 77 inches apart), exhibited a (calculated) extension of fibre of .00573† and a co-efficient of elasticity of about 40,000,000 pounds per square inch; and at the lesser flexure of ¼ inch (the only other observed), the co-efficient was about the same.

The variation of the co-efficient of elasticity under different degrees of extension, seems to me to be proof positive that in a bent beam there cannot be "uniformly varying stress"—for while the extension of fibre varies, as it must do, nearly uniformly from the neutral axis to the exterior, the co-efficient of elasticity, the constancy of which is essential to a corresponding uniformly varying stress, is, in wrought iron, far from being constant. The co-efficients for cast-iron vary much less than for wrought iron—and indeed, my experiment just cited, with a cast-iron beam while developing an "extension" of .00573, which is nearly four times as great as the maximum extension in the table of Hodgkinson's results, given by Morin‡ .00155, indicate at that extreme extension the same co-efficient nearly, as for the less extension of .00382. Hodgkinson (as quoted by Morin) gives almost identical co-efficients of elasticity for cast-iron for compression as for extension: hence it may be presumed that the same is the case in the cast-iron beam ruptured by me.

There is, therefore, in this important experiment—the largest beam, I believe, ever ruptured experimentally—some confirmation that, for cast-

* Transactions XIV.

† On page 5 of the paper referred to, there are errors in the printing of the figures for the "extension of fibre." They should be .00382 and .00573.

‡ "Resistance des Matériaux." Morin.—1855.

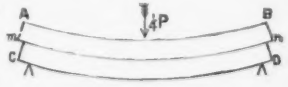
iron, Barlow's theory is true, viz.: that the stress in the cross section *is* a uniformly varying one, and that the neutral axis "traverses the centre of gravity." Yet from this experiment we deduce a discrepancy between the "modulus of rupture" and the "tensile strength" of the iron, even greater than for the case of the wrought-iron beam in which enormous variation in the co-efficient of elasticity is clearly shown. The modulus of rupture for the 12 by 15 inches cast-iron beams would be, in one case, nearly double—in the other about 1.5 times the "tensile strength."

Mr. Barlow is probably right in the opinion (as stated by Rankin), "that the curvature of the layers of the beam produces a peculiar kind of resistance to bending, distinct from that which arises from direct elasticity." It is evident that the theory of a horizontal neutral axis, however convenient, cannot be a true representation of the phenomena of the resistances developed in bending a beam.

Let us suppose the solid rectangular beam *A*, *B*, *C*, *D*, Fig. 7, resting upon supports at extremities to be bent to a *given deflection* by a force *P* applied at the middle. The neutral axis along the middle (according to theory) would be the line *m n*, and moreover the entire horizontal section *m n* is in a rectangular section (or other symmetrical section above and below this plane), reckoned as neutral. But it is evident that it cannot be strictly so—evident that to stresses transmitted across it the three-fourth's computed resistance to flexture is due. For, let the beam be split along the section now, it will become two beams, of which the contiguous surfaces *m n* will slide over each other, and, the flexure being maintained the same, the two halves will appear as in Fig. 8, and the force to produce the flexure will be but $\frac{P}{4}$. There cannot, then, be a neutral line or plane along *m n*, and it may be doubted whether there can be really neutral lines or planes *anywhere* in a bent beam.

Fig. 7.

Fig. 8.



Researches with a perfectly elastic material, as glass, and one, too, which can be made to reveal to the eye indications of its internal strains, cannot but throw light upon one of the most difficult problems in mechanics, viz.: the internal strains of solids subjected to the action of external or internal forces. The general problem can be stated mathematically; but the analytical solution, except for a few simple cases,

defies mathematicians. That portion of Mr. Nickerson's researches concerning the strains in hollow columns is very interesting, and I am inclined to regard his views on the "periodic" character of the strain, and upon the means of counteracting it, as well founded.

Mr. E. A. Fuertes.—Some time ago I noticed a flare or wing-like streak of light when examining the image of a star through my equatorial telescope. Fearful of some accident having taken place and impaired the object glass, and not being willing to remove it from its cell, I held in front of it a spirit lamp, on the wick of which I had poured some common salt. I at once observed the formation of Newton's rings on the lenses, but near the edge of the cell holding the lenses, the rings, instead of being concentric were scalloped, or bent inwards towards their centre. Remembering experiments made by Pasteur, upon the pressure of mercury over a thin glass surface, I further examined the cell, and found that the disturbance of Newton's rings occurred near one of the screws which held the cell on the barrel of the telescope. I loosened the screw, and, to my great and natural delight, the interference suddenly disappeared. It was then evident that the cell had contracted through cold since the time at which it was screwed in the telescope, and that the pressure produced by this contraction caused the imperfection of the image, the location and extent of which was revealed by the soda light.

Gen. T. G. Ellis.—Several years ago I gave much thought to the subject of strains in beams, and regretted that those who had given their attention to experimenting, had pursued their investigations in so unscientific a manner. Mr. Nickerson's examinations are a step in the right direction, although I do not see that he has developed much more than has heretofore been done by Wertheim of Paris, who invented an instrument for the determination of strains and pressures by the colors of polarized light. Prof. Tyndall has also done the same. I sincerely hope that Mr. Nickerson will pursue the matter further. What we want to know is the elastic resistance to tension and compression in any one substance, as glass, and the effect of a weight upon a beam of the same substance. Then, knowing by experiment the elastic resistance in both directions of other substances, we might reason with some hope of success upon the position of the neutral axis of beams of different substances. In my opinion a great error of writers on the strength of beams is, in assuming that equal strains will elongate and compress the same substance to an equal amount, thereby making the neutral axis of the

horizontal strains in the middle of a rectangular beam ; also in assuming that the elastic resistances are proportional to the ultimate resistances of the material.

One part of Mr. Nickerson's experiments that I do not clearly understand, is, how polarized light can show a neutral axis in a glass beam supported at the ends with a weight in the middle. It seems to me that it should show only a neutral *point* in the middle of the beam, under the weight. There are diagonal strains in all parts of the beam except this central point, and diagonal strains should transmit the polarized rays as well as the horizontal ones. That there are diagonal strains in all parts of the beam, although they may be very small in the upper end corners, may be shown by supposing the whole to be divided into very thin horizontal laminæ. The flexure of the beams by the weight would then cause all of these to slip one upon the other. This is resisted in the solid beam by the diagonal resistance of the material. The additional element of strength referred to in the paper, is, I apprehend, not due to the absolute resistance of the material to the sliding of the laminæ upon each other, or the "horizontal shearing strength" as it has been called, but to the elastic yielding of the fibres of the material, which, without diminishing their ultimate strength, allows them to slip, so to speak, upon the next interior fibre, so that the latter are drawn to a greater tension, or subjected to a greater degree of compression than they would be if there were no such lateral elasticity between the fibres. The outer ones would be ruptured as soon as their limit of elasticity was reached, and before a full strain was brought upon any of the interior fibres, were it not for this action.

Gen. J. G. Barnard.—I am disposed to think Barlow's position untenable, that the neutral axis *cannot* be in the centre of gravity, when the breaking strain is in excess of the result given by formula founded on that assumption. There is no more radical law of mechanics, than that the forces across any section must be in equilibrium. Take a cross section of a beam, and these forces cannot be in equilibrium if the neutral line is through the centre of gravity when the breaking force is found to exceed that due to tensile strength of the iron.

LXXXIII.

METHOD PURSUED IN REPLACING A STONE PIER ON A PILE FOUNDATION.

A Paper by J. Albert Monroe, C. E., Member of the Society.

Read March 4th, 1874.

The stone piers for the bridge of the Massachusetts Central R. R., over the Connecticut river at Northampton, Mass., rests upon a timber foundation, consisting of piles and grillage. The piles were cut off 2 feet above the bed of the river. The grillage, consisting of 3 courses of 12-inch timber, was placed in position over the piling, and the masonry commenced, being lowered as the work progressed, by means of screws, in the ordinary way, until the grillage rested upon the piles. The grillage for pier No. 8 was prepared on the shore during the winter of 1872-3, and, after a canal was cut through the ice, it was floated into position sometime in January, and the construction of the masonry at once entered upon. The work proceeded with proper dispatch and with no drawbacks until 2 courses of stone were laid, 2 feet thick each, when suddenly the screws let go their hold upon the grillage, and the entire work disappeared in about 18 feet of water.

The false works and various obstructions in the river caused the

formation of whirls and counter-currents. A careful study of the cause of this mishap led to the belief that the ice which had been cut out for the canal and at the site of the pier, and which, as the easiest way of getting rid of it, was shoved under the stationary ice, had been taken up by these whirls and thrown against the grillage with sufficient force to bruise and weaken the horns to which the screws were attached, which conclusion was confirmed by the subsequent examinations of a diver. Operations on this pier were then suspended until the season of low water, in the summer of 1873.

A careful examination showed that the masonry had not changed its position on the grillage, nor had it sustained any injury. The grillage rested upon the piling very nearly in its proper position, up and down stream, but in its descent it had moved easterly, so that its upper end was about 13 inches out of place, and its lower end about 14 inches. The question then arose, between the contractor, Mr. Smith of Springfield, and the company's engineer, Mr. Frost of Boston, as to whether to proceed with the work and finish the pier in that position, or to move it to the position in which it was first intended to place it. It was finally decided to leave the matter to referees, of whom the writer was one. The referees met, and it was determined to let it remain where it was, if, after thorough investigation, it was found that the grillage rested upon a sufficient number of piles to support the structure—providing the piles were properly distributed.

A diver was procured and a complete survey made, which resulted in the referees ordering the grillage and masonry to be moved into the position first intended for it. This decision having been arrived at, it became a question whether it would be more expedient to take up the entire work and commence anew, or to endeavor to move it bodily into position. The latter course was determined upon, after discussing various methods for accomplishing the object.

A plan was adopted, the details of which were worked out by Mr. Smith, and, on September 10th, the mass was moved successfully into place in 7 hours. The pier is 12 feet 6 inches wide, and 42 feet 7 inches long on the bottom, and when moved was 4 feet high, resting on a grillage 3 feet thick, making a total height of 7 feet, against which was quite a strong current, striking it obliquely at an angle of about 35°.

The method employed was substantially as follows; a framework of piles, heavily capped, one side of which was 3 inches higher than the other, to facilitate the movement of the rollers, was first constructed

around the pier. Across this, and over the pier, were placed timbers, in pairs, equal in number to the number of screws employed. Above each of these pairs was a single stick of timber, from which the screws for lifting the pier depended, the screws passing down between the two timbers first mentioned. Between the sticks of timber in pairs and the single stick was a set of rollers, and between the rollers and the timber were iron plates. Other timbers, to which was fastened railroad iron to strengthen them, and at the same time to counterbalance the buoyancy of the water (which it was necessary to do in order to facilitate the handling of them under water), were placed by a diver under the grillage, and between the rows of piling. The screws were attached to these timbers by means of yokes. When the preparations were completed, the screws were turned until the grillage cleared the piling, when, by means of the ordinary iron clamps used in the construction of bridges and similar work, the pier was moved into place.

The principal feature of this operation is, that it was accomplished without the aid of any tools or material other than such as were at hand and on the work, excepting the diving apparatus which was necessary in order to get the timber under the grillage. The framework and screws were the same as employed in lowering the masonry of all the piers; the timber used under the grillage, and that used in connection with the rollers was procured for the floor-beams of the bridge.

LXXXIV.

EUROPEAN RAILWAYS—AS THEY APPEAR TO AN AMERICAN ENGINEER.

A Paper by W. Howard White, C. E., Member of the Society.

Read May 20th, 1874.

A railway built upon the American idea of first getting the line built on a cheap system, and then bringing it up to first-class rank as the receipts permit, is a thing almost unknown in Europe. In but one case, the writer noticed a road being built upon the economical plan, with wooden bridges.

One thing that strikes an American curiously is the immense amount of trouble taken by engineers in laying out work for contractors. The immense number of all sorts of signals, grade posts and profiles, the latter not only for banks and cuts, but even for borrow-pits, would render nearly double the number of engineers necessary with which an American road is built.

In making banks, it is the practice to remove the surface of the ground, in case it is either of bad material, or where it furnishes sod for covering slopes, which are almost always either sodded or carefully planted with grass seed, producing undoubtedly a great saving in the after expense of cleaning ditches in cuts. Upon one road in the Tyrol, a curiously favorable combination of circumstances was seen in a low bank across a meadow, formed from side borrows. The sod from the base of the bank was just sufficient to sod the slopes, and at the bottom of the side borrows was found a nice layer of clean gravel for ballast.

Tunneling has been carried to a point in parts of Germany—notably in the Brenner Pass in Tyrol, and on a new road in the Black Forest—the equal of which would be hard, if not impossible, to find in other countries. In one place on the latter line, the engineer having run his grade into the bottom of the valley he was climbing, boldly turned

somewhat more than a quarter circle directly into the hill—instead of crossing the valley—and came out again on the same side of the valley, above his previous line. At another place he made a curve of somewhat more than 90° underground, and then reversed for 30° before coming out again into daylight. The plan of this Black Forest line is a curiosity of itself. It presents two complete S's, in the necessity the engineer was under to get the distance for a certain grade, and there are 25 tunnels in a distance of less than 17 miles, ranging from mere bridge arches in point of length, up to the summit tunnel, which is over a mile long.

Tunnels are made in many cases, where, with us, they would be decided extravagances; but the circumstances are so different—what with the greater cost of the land, and the less cost of labor—that it is impossible as a mere looker-on to judge them. European roads use a greater depth of ballast than is common with us, though perhaps no more, except in rare cases, than is usual on our first-class roads, 18 inches being about the average. The profile of the ballast is generally level or nearly so for the whole width of the road-bed, the entire reliance for drainage being upon the porosity of the material used. German roads, including those of Austria (of which the practice is in the main the same), ballast up to the top of the tie; the English, half-way up the web of the rail, leaving at every third or fourth tie a cross-gutter between the ties; while the French ballast up to the bottom of the top rail-table, rounding the ballast on the inside of the track, however, so as to carry it down to the bottom of the splices at the rails, while remaining of the full height at the centre. The latter practice deadens the noise considerably, but is of course very inconvenient for "surfacing" track.

One is struck even in Germany, where railways are built in a more economical manner than in France or England, with the extravagance, as it seems to us, in certain ways, such as the use of skew bridges, for instance. In a short length of road, over an insignificant stream, one will see two or three skew bridges, where the stream could easily have been turned, to the avoidance of the extra expense. Another favorite and useless feature are the double-deck stone bridges to be seen in various parts of Europe, notably on the road over the Lemmering Pass, not far from Vienna. The engineers of these bridges seem to have been afraid of having the piers shaken down by the motion of the train, and have consequently built their bridges in two tiers of arches, the lower tier serving the imaginary object of preventing a flexure of the piers in

the axis of the bridge, and having the disadvantage of detracting very much from lightness and grace.

The favorite iron bridge in Europe, even at the present day, is the lattice truss, with larger or smaller lozenges between the pieces. Wrought-iron is used almost exclusively—except for arched bridges—both top and bottom chords being built up of thicknesses of plate, with a double web at the bottom and top of the upper and lower chords respectively, between the plates of which the ends of the lattice are inserted, and thus secured by riveting to the chords.

The rail sections used on the continent are similar in the main to our own, but much heavier as a rule, and also higher in proportion to the width of the bottom flange, the modern German standard rail section weighing from 75 to 85 pounds per yard, and being from $4\frac{3}{4}$ to $5\frac{1}{4}$ inches in height, with a base of about 4 inches, which weights and proportions are also about those of the most approved French practice. English rails are of about the same weight or a little heavier, but they are almost universally of the double-headed pattern. France has still a number of lines laid with double-headed rails, but the practice inclines to the other. The ties upon the continent are about 3 feet apart. Our practice of placing the ties nearer together equalizes somewhat the lighter weight of our rails.

European rails are generally symmetrical, the inclination for the coning of the wheel being given upon the tie; but in two cases of roads in Germany where new rails were being laid, the writer observed rails going down with the inclination in the head itself, which is, perhaps, an indication of the modern tendency. The more ordinary position on the continent for the notch to receive the spike holding the rail in position longitudinally is the middle of the rail, but since it has been so thoroughly proved that these notches diminish the strength of the rail to a great degree, it is hardly probable that this practice will continue in spite of its convenience as compared with notches at the ends.

The split switch so commonly used upon our western roads is almost the only one used in Europe. The switch lever has no locking apparatus, but is kept in place by a counterweight upon a short movable arm nearly at right angles to the lever. The swinging of this arm a half circle around the lever brings the weight into place to hold the switch in one or the other position. This arrangement has the advantage of flexibility, so that the "running through" of a switch by a careless engine-driver does not throw any severe strain upon the ap-

paratus. It would perhaps be hardly safe to apply it, except where, as almost universally in Europe, the switchman is constantly on hand to see that everything is right.

The gauges throughout Europe are, with few exceptions, 4 feet 8½ inches. The most notable are of Spain and Russia, the former being 5 feet 8½ inches, and the latter, 5 feet.

In Germany, the first item in the way of signals that strikes an American is the electric bell system used for communicating information or instructions of simple character to the watchmen who are stationed at more or less regular intervals—generally at level crossings—along the line. Each watchman's station is provided with a gong of large size, mounted upon a little building—or rather, upright box, containing the mechanism. The gongs, by the way, are generally double, and the notes are made harmonic, being said to be more effective for hearing than single bells. Upon all of these bells between any two stations, signals are made simultaneously by means of hammers striking the bells, operated from either terminal station. The signals given are few in number, the chief ones being the departure of a train from one or the other station, signaled by a different number of strokes. The effect of the striking of these bells is sometimes very pretty, particularly when heard from a high point, where those at several stations can be heard striking at the same time, the strokes coming to the ear, however, at different instants

Many roads upon the continent now use an electric switch signal at either end of a station ground. This signal stands normally at danger. When, however, a train is expected and everything is clear, the switchman sets the signal at "all right," whereupon an electric bell is set in motion at the station, which rings until the train has passed, and the watchman has released the signal, which goes back to "danger." It will be noticed what a safe signal this is. Since it stands ordinarily at "danger" no train can enter the ground until the signal is set. If either the signal is not set, or the apparatus has fallen out of order, the station-master, not hearing the bell in action when the train is expected, investigates the matter. If, on the contrary, the signal does not go to "danger" after the train has passed, or if the apparatus gets out of order, so that the bell continues to sound, the station-master is equally warned. This apparatus is used upon a few roads with us, the Eastern R. R. of Massachusetts for one since the unfortunate collision at Revere, some years ago.

A little convenience for superintending engineers, road-masters, and

others interested in knowing the rate of grade at different points of the line is to be found in the grade posts set throughout Europe, at the points of change, consisting simply of a post, rising any convenient height above ground (most commonly only a foot or two), with two pieces of board fastened to it like the arms of a sign-post, which are, however, inclined to show the direction of the respective grades, and are marked with the rates of grade, and generally with the lengths of the respective inclines or levels.

Signals and crossing-gates, the latter generally being simply bars like old fashioned well sweeps, held open by a counterpoise on the short end, are operated at long distances, in some cases (at different ends of a tunnel for instance) as great as a mile or more, by wires running upon little rollers at regular intervals; a system which I believe is only to be seen upon our side of the water, upon Canadian roads, unless in some exceptional cases.

Traversing and turn-tables for transferring freight cars from one track to another are very much used; every station being provided with at least one transverse line of rails for connecting the various tracks. The transverse track on the same level with the main lines, and having turn-tables at its intersections with them, is the most popular, and is a decided convenience where a large staff of employes is on hand to move cars, but the much greater length of our cars, and the consequent increase in the expense of such apparatus will probably prevent its being adopted extensively in the United States.

There is one use of traversing tables, however, which has not fallen under the writer's observation in the United States, and which seems a very useful one in cases where economy of space in engine-houses is a consideration; that is, what is called the rectangular system. As practiced in France, where chiefly used, it consists in arranging the engine stalls upon parallel tracks in a building divided in the centre by a pit, with a transverse system of rails, carrying a transfer-table. Each of the parallel tracks gives two stalls, lying upon either side of the transfer pit, whose table, generally provided with a small steam-engine, gives the means of moving an engine from the entrance track to any stall, and vice versa. Under the system as practiced abroad, the pit is covered, which involves a great expense of roof over waste space—the chief defect of the system. If snow should not prove too great an obstacle, there would not seem to be much reason otherwise why the pit should not be left open, dividing the house into two separate ones. The system

offers considerable advantages over lunettes or round-houses for repair engine-houses, in the greater facility it affords for setting up shafting or traveling cranes.

The lightness of the running gear of European rolling-stock strikes the American observer. Freight cars weighing but 10,000 pounds, carry 20,000 or even as much as 30,000 pounds; and although the proportion is generally far from being so favorable in the passenger stock, it is on an average, perhaps, even more favorable in comparison with our own. Some of the double-deck cars, which are quite common upon French roads, exhibit a most extraordinarily small proportion of dead weight. One on exhibition at Vienna, with a capacity of 90 persons, weighed only 11.75 tons. It would seem these cars might be made even lighter, were it not for the side buffing system, which throws heavy diagonal strains upon the floor frames. The secret of the thing must lie in the superior average character of European permanent way, and it is an interesting question, if, upon the great roads with us whose road-bed and tracks are nearly or quite as good as those of Europe, it is not possible to make some radical reforms in that matter, at least in passenger stock, which is not liable to wander upon badly kept roads.

The wheel-coning in Europe is generally $\frac{1}{20}$, the tendency being rather towards increase than diminution, some roads having made it as high as $\frac{1}{16}$, and $\frac{1}{17}$ having been found to give good results. The Vienna & Salzburg line tried the experiment of doing it away altogether, but it is conceded that the results are bad.

Brakes in Europe are seldom applied to more than a limited number of vehicles in a train. The French "Direction of Roads and Bridges" has declared the Westinghouse brake too complicated in its connection between cars to render its adoption probable upon French roads; a dictum rather astonishing to any one who knows how infinitely more simple the connection is than the ordinary European car-coupling, which involves the giving of at least half a dozen turns to a sort of vice-handle which has to be released and seized again at each turn.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

LXXXVI.

RETAINING WALLS.—AN ATTEMPT TO RECONCILE THEORY WITH PRACTICE.

A Paper by CASIMER CONSTABLE, C. E., Member of the Society.

PRESENTED APRIL 20, 1874.*

PRELIMINARY.—The stability of a retaining wall is expressed simply by the equality of two moments; the weight of the wall multiplied by its lever arm, and the thrust of a certain triangular prism of earth (or material behind the wall) multiplied by its lever arm; the point of rotation or fulcrum being the intersection of the line of pressure or "resultant thrust," and the line of rupture of the wall. For exact equilibrium, these two moments are of course equal. In practice, however, a factor of safety should be used, for similar reasons as it is used for iron in bridges and other structures. Many tables and formulas, however, are presented for use, where no factor of safety is introduced.

For some time I have collected instances of light walls which continue to stand well, and yet are as light, and in a few instances lighter, than generally accepted theory requires. The reason for this I will proceed to set forth, and illustrate it by the model presented. It may be well, before doing so, to admit that often there are cases of walls, far stronger than theory calls for, which have failed, or started to give way. I think the explanation is simple; the failure, from an improper judg-

* Condensed from a Paper read January 22d, 1873.

ment connected with the construction, is due to imperfect backing and similar causes, to provide against which, an extravagantly large factor of safety was employed.

In carrying up the backing, and providing for proper drainage (through "weepers" in the structure, or otherwise), lies the greatest opportunity of practicing economy in the construction of retaining walls. If greater care were taken in this respect, the engineer would find more need than he now does for formulas in the design of walls; safe, and at the same time small factors of safety might then be employed.

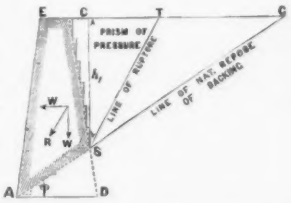
The following example was furnished by a well-known member of the Society. The wall, of which Fig. 1 is a section, was built to support a wharf for receiving pig iron. It was started on a level bottom, at the foot of a slope of rock (the surface of which was quite smooth), laid dry, and backed as the work proceeded. When the wall was 7 feet high, a load of iron dropped on the backing, caused it to bulge out. Evidently the prism of earth pushed with the force of a wedge, moving with little friction along the face of the rock. The wall was partly taken down, and at a small expense, the rock was stepped, and the earth carefully rammed, as was first suggested by the engineer. When this was done, the wall was completed, and therefore considerably lightened in proportion to its height, yet it stands a load of 55 tons of iron to the running foot on top of the backing. Some of this great weight undoubtedly does not rest on the prism of earth which presses against the wall, but now the wall has a thickness of only 0.26 of its height, while at first it gave way with a relative thickness of 0.66. This shows the great importance of careful construction, without which theory will avail but little.

Before the equation of moments can be calculated, the conditions of the problem must be determined. It often happens that data is assumed, and generally accepted without question; in such instances, theory, however carefully worked out, will fail to agree with the facts in practice. Now, although most of the conditions of this problem are established, the writer noticed, in the case of a wall whose end was exposed, that, on falling, the entire wall did not overturn, but sheared off at an angle of about 45°, starting at the foot. The experiments tried in consequence, subsequently, showed this was not accident. One condition is thereby changed, namely, *the "prism of pressure" must rise from a point above the foot of the wall.* Hence, in the formulas to be deduced, the weight of the prism of earth will be less, and other conditions remaining the same for equilibrium, the weight of the wall must be lessened; that is,

a lighter wall will suffice for exact equilibrium, and there follows, at least in part, a reason why walls, apparently too weak, have stood well.

Height of Prism of Pressure.—It has been generally assumed that walls turned over as a solid body, rupturing along the bed, *A D*, Fig. 1. This has been shown not to be the case, but that the wall breaks along a line, *A S*. The height of the prism of greatest pressure then becomes *C S*. The angle at which the masonry ruptures appears to be about 45°, the the angle of repose of broken stone; but to err on the safe side, in the absence of perfect data, let the angle of repose of the backing material be taken, then the line *S C* will be 0.75, the height of the wall. The other conditions are unchanged, and, by taking the moments about *A*, a formula is easily deduced which will give the required thickness of any wall. For unsurcharged walls it is as follows:

Fig. 1.



$$t = h \times \left[-\left(n + \frac{n_1}{2}\right) \pm \sqrt{\frac{2 \tan.^2 \frac{\alpha}{2}}{7\ w}\left(w_1 + \frac{8}{3}\ p\right) + \frac{n^2}{3} - \frac{{n_1}^2}{12}} \right] \quad [A]$$

in which t is the thickness at top, h the height of wall, n the batter of outside, and n_1 that of inside, in decimals of a foot per foot rise; α the angle of repose of the earth with the vertical $= 90° -$ angle of repose φ, w the weight of a cubic foot of masonry, and w_1 of earth, and p the weight per square foot of surface which may be on top of the earth.

If the wall is rectangular in section, n and $n_1 = 0$, and

$$t = h \sqrt{\left(w_1 + \frac{8}{3}\ p\right) \frac{2}{7\ w_1 \tan. \frac{\alpha}{2}}} \quad [B]$$

If there is no load, as of a building or train, the formula becomes simply:

$$t = .53 \tan. \frac{\alpha}{2} \sqrt{\frac{w_1}{w}}. \quad [C]$$

Had it been assumed that the prism started from the foot of the wall, or the wall turned bodily over, the formula would have been:

$$t = .57 \tan. \frac{\alpha}{2} \sqrt{\frac{w_1}{w}}. \quad [D]$$

which will be recognized as the usual formula. The difference is but about 8 per cent., but this is no small quantity in a long wall.

I had a box made, 16 inches high, 24 inches long, and 16 inches

wide, with glass sides; also a number of blocks of pine, representing bricks, 1 inch square, 2 and 3 inches long. In eight experiments with walls, 3 and 4 bricks thick, stuck together with water to prevent sliding on the joints, and fastened by pin points to prevent sliding on the base (in consequence of the smooth surface of the pine), the walls turned over as in Fig. 2, usually breaking at an angle of 45°. The backing material first used was oats, the angle of repose being 30° with the horizon, the specific gravity very nearly that of the pine bricks, while the cohesion due to the length of grain resembled that of many earths. Any tendency to overturn was carefully noted with a square, by a scale drawn at the foot of the wall on the bottom of the box. A slight play was allowed between the sides of the glass and the ends of the wall.

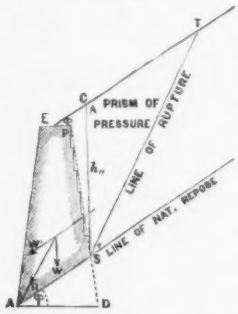
Continued stability was indicated by the absence of any reaction when an irregularity in the face of the wall due to building up was corrected by pushing it back with the square. When the wall began to give, it bulged, and would not, when pushed back, remain so. The centre of the curve appeared to be the middle of the height. Owing to the cohesion of the oats the walls gave way gradually, and when leaning over would continue in that position till started forward by a jar. This tendency, which for simplicity and want of experimental data, is omitted in theory, is with fresh earth insignificant, but in walls of long standing, where the earth has become well consolidated, undoubtedly adds to their stability. In practice the fact is often noticed, that a wall, having started forward, remains in an apparently threatening position for years. When a solid wall was substituted for the one of blocks, with a joint, stepped, as the brick had separated, along $A\ S$, this feature increased, although both styles of wall began to fail at the same stage, which perhaps goes to show what is held to be the case by many, that a brick wall of stability equal to one of masonry, is not as strong.

Finding the friction along the sides insignificant, the box was reduced in the width to $12\frac{1}{4}$ inches; the extra quarter inch allowed the wall, which was 12 inches long, to turn without binding, and the extra length of backing tended to counteract any little friction of the same on the glass sides. To make the effect of an increasing height of wall more sensible, a ratio of earth to wall of two to one was adopted, by using peas, the angle of repose being 28°. These were so fluid that, after equilibrium was reached, the walls gave way instantly with a complete overthrow, breaking, however, as before, at an angle, though the rapidity of the fall prevented one at first from noticing the fact, as the impetus

finally acquired by the heavier and more fluid material pushed the remaining triangular portion of the wall forward on its bed joints. After carefully watching several trials, the rupture, however, became clearly visible, and, by dowelling the bed joints with common pins, the triangle of wall remained undisturbed, in spite of the rush of peas over it. If the wall had had a tendency to turn in one solid body from the base, this dowelling would have favored such a result by holding the beds together.*

This shearing of the wall appears most reasonable. Certainly if *A D S* (Fig. 2) were of earth, instead of masonry, it would remain at rest if there was no wall there. Now replace it by the latter, and suppose the wall to give way. Even should the triangle *A D S* begin to raise, then the earth behind it has no tendency to move forward. Therefore the plane of rupture will start from the point *S* in the direction of *T*, and as it is the starting prism alone which we have to account for as causing rupture, *S T C* will produce the pressure which the wall must be able to resist.

Fig. 2.

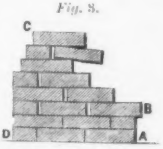


The walls built of these brick blocks, and afterwards made solid, except at the breaking joint, were tried many times, to be sure that the weight at which they gave way was correct. The reaction proved most sensitive, so that the smallest possible increase in weight could be noted. This, in the experiments, was done by adding thin pieces of the width and length of the wall, and one inch high, by the miniature scale. Finally the walls were proportioned by Formula *C*, and the following examples will show how nearly they agreed. When the

Height of Walls.	Thickness. By Experiment.	*C.*	*D.*
5	2½	2.28	2.46
7	3	3.15	3.44
10	4	4.40	4.90
12½	5	5.51	6.00

* Since writing this Paper, the writer found, in the Transactions Royal Engineers, 1845, an account of some experiments, tried with actual brick walls, for a special purpose. Some of these were carried to a height of 20 feet. A careful section after rupture was given, but no conclusions of a theoretic nature were drawn. As no better proof could be produced than this of what I have advanced, and as the paper is out of print, and rare, a copy from it is submitted. Fig. 3 is a section of the walls as they gave way. No mortar was used between the bricks, which explains their sliding somewhat on their beds.

Fig. 3.



angle of rupture was taken at 45°, instead of that of the backing, the agreement, as might be expected, was much closer. In the table annexed, column C is calculated by the formula, and column D by the old formula. Column C, is still larger than found necessary, by experiment. This is chiefly due to the line of rupture being assumed in the formula to be the line of natural repose of the earth, whereas it was at an angle of 45°. Also $\frac{w_1}{w}$ exceeds what is usual in practice.

Surcharged Walls.—In the case of surcharged walls, overturning takes place in the same manner, and the formula, when modified, gave closer results than that for ordinary walls. The experiments were repeated many times, and the walls just gave way when they reached the calculated height E.

In the preceding case the surcharge slopes off indefinitely. It has been given to cover a limited surcharge as well, since the formula, in such a case, become too unwieldy; and under extreme limits the difference in assuming a "limited surcharge," indefinitely prolonged, is not more than 3 per cent. In every experiment tried, the results were most satisfactory, the heights calculated by Formula E being those almost exactly at which the walls gave way. The experiments were all repeated several times, so that there might be no mistake.

In the absence of a table of cosines, the following may be of use in an engineer's note-book. It is carefully calculated from Formula E:

	No Factor.	Factor of Safety = 2.*					
$\frac{w_1}{w}$ =	1.0	1.0	0.9	0.8	0.7	0.6	0.5	0.4
Angle of repose, with horizon.				$\frac{t}{h}$				
280	.3355	.470	.445	.418	.395	.364	.330	.300
300	.3291	.461	.440	.412	.387	.357	.327	.295
320	.3222	.451	.429	.403	.380	.350	.320	.283
340	.3150	.441	.420	.394	.370	.342	.313	.278
360	.3074	.430	.410	.384	.360	.333	.305	.270
380	.2994	.419	.398	.375	.350	.325	.300	.265
400	.2911	.408	.388	.365	.340	.316	.290	.257
420	.2823	.395	.375	.353	.330	.306	.280	.250
440	.2732	.383	.364	.342	.320	.297	.270	.240
460	.2640	.370	.352	.330	.311	.287	.263	.233

* That is the earth prism is assumed at double its real value, when in equilibrium—hence Column 3 is Column 2 multiplied by $\sqrt{2}$.

Transformation of Profiles.—Vauban's well known rule for transforming a rectangular wall into one of equal stability, with a batter on the face, seems to have been little used, yet it enables us, in calculation, to deal only with rectangular ones for which the formula becomes simple. It is correct to within $\frac{1}{120}$th, so long as the batter exceed $\frac{1}{6}$th. In practice, now, it is usual to give a less batter; the error then becomes only $\frac{1}{71}$st. I find, by taking the common thickness at $\frac{1}{10}$th the height instead of $\frac{1}{6}$th, this error is very much reduced, provided small batters, as $1\frac{1}{2}$ inches per foot, are used.

By ordinary statics $W = 1.16\ w\ t\ h_1$; $W_1 = \dfrac{w_1\ h_2{}^2\ cos.\ \varphi}{2}$; the lever arm of W is practically $\dfrac{t}{2}$, and of $W_1 = \dfrac{h_2\ cos.\ \varphi}{3}$, ($W$ being the weight of wall, and W_1 that of the prism of pressure), the center of moments is at A; hence

$$\frac{1.16\ w\ t^2\ h_1}{2} = \frac{w_1\ h_2{}^3}{6}\ cos.{}^2\ \varphi.$$

If the surcharge start from the back of the wall, then $h_1 = h_2$, and remembering $h_1 = 0.70\ h$,

$$t = 0.38\ cos.\ \varphi.\ \sqrt{\frac{w_1}{w}}\ h \qquad [E]$$

If the prism of pressure starts from the foot, at back of wall, then

$$t = 0.574\ cos.\ \varphi.\ \sqrt{\frac{w_1}{w}}\ h.$$

Here the difference in the formulæ is as much as 50 per cent.

By the model, with peas as before,

$$t = 0.48\ h,$$

whence the calculated thickness of wall is $3\frac{3}{8}$, compared with $3\frac{1}{16}$, as actually required.

The rule is: *revolve the face line to the proper batter, about a point on this line situated above the bottom $\frac{1}{10}$th the height of the wall*, the resulting profile is one of equal stability with the former rectangular one. The correctness of this rule was proved by experiment.

Counterforts.—The percentage of gain, theoretically estimated, has been found so small as to be outbalanced by the extra cost. In some English experiments it was proved that much resistance was caused by the friction and adherence of the earth to the sides of the counterforts, and that, too, when they did not extend in more than 9 inches. They needed, however, to be at frequent intervals, otherwise the wall gave way between the piers.

If the manner in which these model walls give way is examined, the fact will be noticed that the earth or peas do not revolve, as stated by some writers, about the bottom point of the prism of pressure, but that they settle suddenly on and parallel to the line of rupture, and then come to rest until a jar starts them again. Now, stepping the back of the wall so as to increase the friction against it, at the same time reducing the batter on the face on account of moisture, are practical features well worth considering. Engineers hold opposite opinions on the matter.

Factors of Safety.—A constant factor of safety cannot be determined upon, owing to varying circumstances, but in a formula which gives closely the thickness for exact equilibrium a factor can be introduced understandingly, according to the relative circumstances, and subject to the judgment of the engineer. Taking the moment of the wall at twice that of the earth prism has given good results, and does not make the wall too heavy at any time, since, in the formula for the thickness, the factor becomes the $\sqrt{2}$ (or whatever the assumed factor may be) instead of 2.

In conclusion, it may be said, so much depends upon the manner in which the work is done after the profile has been determined, that economy is best attained by a proper regard to the practical details of construction. Too often these are carelessly attended to, and the safety of a structure of ample strength, if properly built, is brought into question.

I am inclined to think that it is more important to lay particular stress on the close fitting of the backing earth than on the use of good cement and mortar, careful bond or large stone. These, of course, are important and very desirable, but they will not save a wall if the earth has shrunk from it, and there is a sudden settlement of the earth wedge.

Mr. J. Dutton Steele.—A practical rule verified by my experience—for surcharged walls of dry masonry less than 18 feet high—is to make them 3 feet wide on top, with a batter of $\frac{1}{4}$th outside and none inside. In one case for a mortared wall, 18 feet high, I reduced the width at top to $2\frac{1}{2}$ feet and made a batter on both sides. Engineers who, from lack of room, have been compelled to lay walls upon narrowed or stepped foundations will be pleased to know, from Mr. Constable's experiments, that such conform to theory, and are safe in practice.

Mr. Francis Collingwood.—Is it not better to step the back of a wall rather than give it a batter?

MR. CONSTABLE—It is more a matter of practice than of theory ; upon thus stepping a wall, the back filling upon settlement does not act as a wedge.

MR. STEELE—Generally now the back of a wall is not stepped, as formerly, but made vertical : often in railway practice it is counter-sloped or underent, and the stability thereby increased. The back should have a "frost" batter at top where the earth is likely to freeze, so that it may be lifted from the wall ; care should always be taken in backfilling to slope the packed earth from the wall rather than towards it.

MR. ISAAC D. COLMAN—In filling behind the masonry of the New York State canal locks, broken stone one foot in thickness was placed between the wall and embankment.

MR. CONSTABLE, by experiment with the model, demonstrated that two walls of the same area of section, with batter on face, of 0.22, one with parallel sides and the other with a vertical back, were equally stable, and also that the saving in material by giving much batter is but little. A wall battered on the back less than on the face, evidently is less economical than if all the batter was on the face.

He submitted a communication from a Canadian engineer, from which is quoted : In practice I have always made my walls heavier than theory demanded, on account of the severe operations of frost in this northern latitude, where it strikes from 3 to 4 feet into the ground ; and yet, without giving a slope or "frost" batter to the back of the wall, when the frozen earth presses against it, our strongest walls could not stand.

It has been my rule to make the base of the wall $\frac{2}{3}$th its height, but this is for first-class masonry, laid in hydraulic cement.

LXXXVII.

NOTES ON THE

FLOW OF THE WEST BRANCH OF THE CROTON RIVER.

A paper by J. James R. Croes, C. E., Member of the Society,

Read May 6, 1874.

The construction of the first storage reservoir for impounding a portion of the surplus flow of the Croton River for use in dry seasons to keep up the requisite supply for New York city, was begun by the Croton Aqueduct Board in 1866. This reservoir is situated on the West Branch of the Croton, in the town of Kent, Putnam county, N. Y., 60 miles north of New York and 23 miles by the course of the stream above the Croton dam, at the termination of the aqueduct. It receives the drainage of 20.37 square miles. The surface of this water-shed is very broken and undulating, the hill sides steep and rocky, a large proportion of the area covered with timber, and of the cleared portion the greater part is kept in grass, very little being cultivated. The rock, which lies near the surface over most of the area, is a very compact gneiss.

Observations of rain-fall have been made since June, 1866. The point at which they were taken until 1872, was 565 feet above the level of the sea, in the valley a mile and a half above the dam. There were of course many showers extending over only a limited area, but the observations taken at this point, afforded, I think, a very fair average of the rain-fall on the entire water-shed. The gauge used was of the "Smithsonian" pattern, 2 inches in diameter.

Measurements of the flow of the stream, near the dam site, were taken under my supervision from April 27th, 1867, to July 13th, 1868, and from September 26th, 1869, to October 1st, 1870. Observations were generally made three times a day (during freshets much more frequently, and care was taken to note the time of maximum flow after rains) until January, 1871.* After that date, only one reading was taken daily, to February 1st, 1872, and from then no measurements were made until April 10th, when they were resumed, and continued until November 8th,

* I am indebted to the Chief Engineer of the Croton Aqueduct for copies of the gauge readings made subsequent to October 1st, 1870.

1872. The total number of observations which have been reduced to obtain the results given below, is 2,510.

The following method was pursued in making the gaugings: all the water which flowed from the drainage area tributary to the reservoir, was caused to flow over a weir with horizontal crest and vertical side boards one inch thick. In the first series of gaugings the weir was 24 feet long, in subsequent ones, 21.15 feet, and this was afterwards reduced to 18.02 feet. The channel just above the weir was 3 feet wider than the weir was long, with parallel sides, planked for 15 feet, above which it widened. The height of the crest of the weir above the bottom of the channel was about 18 inches on the upper side, and there was a clear fall of about 30 inches on the lower side. With this weir and heads of 0.15 to 4 feet, perfect contraction was obtained.*

The head was measured by a float gauge enclosed in a box, and attached to a graduated rod. Ordinarily there was no difficulty in reading this gauge to 0.005 feet. When the float was unsteady, the mean of the oscillations for several minutes was taken. The gauge was placed far enough above the weir to be unaffected by the fall in the surface in passing over the weir.

For all heads above 0.15 feet the discharge has been computed by Mr. Francis' formula

$$Q = 3.33\,(l - \frac{2}{10}\,h)\,h^{\frac{3}{2}}$$

in which Q = cubic feet per second, l = length of weir, and h = head on weir; all in feet.

The water generally approached the weir without appreciable velocity; in heavy freshets there was, however, some current; no account of which has been taken in the computations. It is possible, therefore, that the quantity of water which passed the weir in very heavy freshets, was somewhat in excess of the calculated amounts.

The diagrams I and II show the course of the flow during the months when there was no snow on the ground, for the years 1867 and 1870. They include the periods of greatest and least flow. These diagrams, exhibiting the great fluctuations in the flow during 24 hours, show how little reliance can be placed upon gaugings of a stream which are made only once a day, and how impossible it is to form any idea of the discharge of a stream from a single gauging.

* On two occasions the head exceeded 4 feet for a few hours, and during the dryest season it was occasionally less than 0.15 feet.

During the first series of gaugings (1867–8), the channel was unobstructed. In the later ones, the dam had been partially built and the water before reaching the weir was obliged to pass through two openings of 4½ feet diameter. In ordinary stages of the water the effect of this obstruction was inconsiderable, but in heavy floods the difference was very evident, in the diminution of the rate of increase in the flow when the water was rising, and the longer continuance of the maximum flow, in consequence of the large area of the water stored above the dam.

In storms, the flow began to increase as soon as the rain began to fall, and the maximum flow was reached from 6 to 8 hours after the rain ceased. The maximum flow measured was in August, 1867. On August 1st, the flow was only 28,000 cubic feet per hour. Between 7 o'clock P. M. and noon of the next day, 1.96 inches of rain fell, and by 6 o'clock P. M. the flow had increased to 336,000 cubic feet per hour. Before the effect of this rain had ceased, light rains on August 3d and 7th, and heavier ones on the 10th and 14th kept the stream full. Between 7 o'clock P. M. of the 15th and 10 o'clock A. M. of the 16th, 3.38 inches of rain fell. At the beginning of the rain, the flow was 242,000 cubic feet per hour; by 6 o'clock P. M. of the 16th, it had increased to 2,432,000 cubic feet, an average hourly increase of 95,000 cubic feet. This was the greatest flow observed. The freshet on this occasion was said by residents to be the greatest known for many years. The flow at the maximum was equivalent to 0.05 inch per hour in depth on the whole water-shed.

While this was the greatest amount of water which passed the weir at any time, the actual discharge of the stream was greater in February, 1870. The ground had been covered with snow, when on the 17th, a thaw set in accompanied with rain. In 28 hours, 2.41 inches of rain fell, accompanied in the afternoon of the 18th, by thunder and lightning, the thermometer standing at 53° F. The water came in such quantities that the openings in the dam were insufficient to carry it off, and it rose 20 feet, storing nearly 50,000,000 cubic feet behind the dam. For 24 hours the flow was 2,200,000 cubic feet per hour; making 52,800,000 cubic feet per day. At the ordinary rate of increase and decrease of flow in freshets, I estimate that the maximum discharge must have been about 3,500,000 cubic feet per hour. This would be equivalent to 0.075 inch of rain-fall per hour. In the storm of October 3d to 5th, 1869, which created extraordinary freshets in the Eastern States and did great damage, and of which very full data were collected and published by Mr. J. B. Francis, the rain fall in 60 hours at this station was 5.23 inches. The

greatest flow was 1,255,000 cubic feet per hour. This storm occurred after a very protracted drought, and the ground was uncommonly dry and absorbent.

The maximum flow during any one month was in April, 1870, but the hourly flow at no time exceeded 1,274,000 cubic feet. The stream was uniformly high, from frequent rains and the melting of the snow on the ground. The minimum flow was reached on August 1st, 1870, when only 32,000 cubic feet passed the weir in 24 hours.

The month of least flow was September, 1870, the total discharge of the stream for 30 days being only 4,508,000 cubic feet—about as much as flowed off in an hour and a half during the freshet of the previous February.

Some peculiar fluctuations in the daily flow as shown on the diagrams, deserve attention. It will be noticed that when the stream was falling after a storm, the flow frequently increased during the middle of the day, by several thousand cubic feet per hour, and diminished again during the night, causing curious *jogs* in the profile. This increase was due to a mill at the outlet of the White Pond, about 4 miles above the weir. To run the mill, the water of the pond was drawn down during the day, and the discharge was shut off during the night. On the other hand, in September and October, 1870, when the stream was very low, the flow was greatest in the morning and diminished during the day, increasing again at night. I suppose this variation in the flow to have been caused by the greater evaporation during the day from the surface of the stream and the ponds. The flow was so small during these months that the mill above mentioned could not run. Early in the spring, on days when it froze at night and thawed in the day time, the rise and fall of the mercury in the thermometer were marked by corresponding fluctuations in the flow of the stream.

Diagram III shows the monthly rain-fall from June, 1866, to January, 1874, and also the monthly flow of the stream during the 49 months in which gaugings were made. Table I gives the amount of rain-fall and of water, which flowed off during each month in which the stream was gauged reduced to inches in depth on the entire water-shed, and also the ratio of flow to rain-fall.*

* In 1872, gaugings are wanting from February 1st to April 10th, but as there appeared to be a certain ratio of flow to rain-fall deducible from the observations of the three preceding years for that period, I have applied that ratio to this interval in order to obtain a full year for comparison. It is very evident that no gauging for a single month affords any criterion for judging of the proportion of the rain-fall which can be stored during a year.

Table I.—Monthly Rain-fall and Flow; West Branch, Croton River.

	1867.			1868.			1870.			1871.			1872.		
	Rain-fall in inches.	Flow in inches.	Percentage of rain flowing off.	Rain-fall in inches.	Flow of inches.	Percentage of rain flowing off.	Rain-fall in inches.	Flow in inches.	Percentage of rain flowing off.	Rain-fall in inches.	Flow in inches.	Percentage of rain flowing off.	Rain-fall in inches.	Flow in inches.	Percentage of rain flowing off.
January	.....	.....	.....	2.90	2.346	80.886	4.51	6.682	125.980	3.80	0.674	17.747	1.44	2.687	186.597
February	.....	.....	... .	1.38	0.833	60.340	6.40	6.647	103.859	3.81	2.248	59.013	1.22		
March	.....	.....	.....	2.55	5.331	209.071	3.80	3.912	102.955	4.27	6.744	157.946	2.59	(c) 6.042	125.622
April	.....	.....	.....	3.87	3.868	99.953	5.45	8.630	158.354	3.01	2.806	93.212	1.00 (d) 2.04	(c) 2.690	131.867
May	6.86	5.031	73.345	8.79	7.342	83.528	2.30	2.306	100.265	3.45	3.406	98.722	3.69	1.725	46.748
June	5.28	3.756	71.128	4.53	2.317	51.141	2.06	0.842	40.864	5.73	1.007	17.576	4.00	1.738	43.450
July	5.25	1.649	31.410	(a)1.90	0.316	29.000	3.43	0.463	13.498	5.07	0.370	7.301	4.34	0.594	13.687
August	10.04	5.403	53.819	1869.			5.10	0.174	3.412	5.24	0.805	15.365	5.99	1.546	25.809
September	3.62	1.443	39.850	(b)1.86	0.097	5.193	2.85	0.095	3.344	1.44	0.569	39.521	3.69	8.875	23.713
October	3.66	1.093	29.869	9.46	5.328	56.324	4.73	0.531	11.233	6.18	1.761	28.490	2.15	0.554	25.767
November	3.10	1.757	56.668	2.43	2.667	109.753	2.51	1.017	40.506	4.35	3.370	77.467	(e) 0.10	0.110	110.181
December	2.62	3.690	140.800	5.96	5.740	96.304	1.49	0.978	65.617	2.58	2.201	85.322	.....	.....	.....

(a) Part of month, July 1st to 13th. (b) Part of month, September 26th to 30th. (c) Flow estimated. (d) Part of month, April 9th to 30th.
(e) Part of month, November 1st to 6th.

In 49½ months, total rain-fall 206.25 inches; flow, 129.764 inches; percentage flowing off, 62.916.

In arranging these results for comparison, I have, for reasons given hereafter, assumed the year to begin with November 1st. This gives three full years consecutively. The first series of gaugings not affording a year beginning at that date, the one beginning with July 1st is taken. Table II gives the yearly proportion of flow to rain-fall for the years as stated above, and also for the 12 months of greatest and least rain-fall and flow.

TABLE II.

Year ending	Rain-fall—inches.	Flow—inches.	Per cent. of rain flowing off.	Loss by evaporation, &c.—inches.
July 1, 1868	52.31	37.071	70.866	15.239
Nov. 1, 1867	52.12	.....	.....	.....
Nov. 1, 1868	49.05	.....	.....	.....
Nov. 1, 1869	46.97	.....	.....	.....
Nov. 1, 1870	49.02	37.689	76.886	11.331
Nov. 1, 1871	46.00	22.385	48.664	23.615
Nov. 1, 1872	39.08	24.082	61.623	14.998
Nov. 1, 1873	50.61	.....	.....	.....
Mean of 8 years	48.00	.....	.....	.....
Mean of 49½ months [a]	50.00	31.458	62.916	18.542
Oct. 1, 1870 [b]	53.75	42.663	79.373	11.087
May 1, 1870 [c]	39.36	18.879	47.964	20.481
Mean of 1870, 1871 & 1872 [d]	44.70	28.052	62.757	16.648

From June, 1866 to January, 1874, the greatest rain-fall in any consecutive 12 months was 56.41 inches for year ending March 31st, 1869, and the least was 38.38 inches for year ending September 30th, 1869. Unfortunately these extremes were reached during the time that the gaugings were suspended. It will be observed that they were comprised within a period of 18 months. A similar period of maximum and minimum rain-fall occurred within 19 months, when gaugings were taken continuously. For the year ending September 30th, 1870, the rain-fall was 53.75 inches, while for the year ending April 30th, 1871, it was 39.36 inches. The flow during the 19 months was 57.48 inches, being 74.3 per cent. of the rain-fall.

a Mean of all gaugings. *b* 12 months of maximum flow.
c 12 months of minimum flow. *d* 3 years continuous observations, including great drought.

The observations for the three years ending October 30th, 1872, cover a season of extraordinary drought. The rain-fall was much below the average, but the percentage of flow was virtually the same as for the whole period covered by gaugings. The rain-fall for the following year was much greater, and the flow must have been greater.

In calculating the amount of storage room required, it is necessary to consider the use which is to be made of the reservoir. If it is to be used merely as a supplementary reservoir (as is the one on the West Branch), for supplying a deficiency in the flow for a portion of the year, the storage year may be considered as beginning on November 1st, at which time the minimum flow has passed, and in ordinary circumstances the yield of the larger water-shed has increased sufficiently to furnish the requisite supply. From November 1st to July 1st, all the water can be stored, and the whole supply then drawn off in the four dry months following. If, on the other hand, the storage reservoir is to be used also as a supply reservoir, and there is to be a constant draught upon it of the average monthly supply, the year must be considered as beginning on March 1st, as it is not until that time that the yield of the stream increases beyond the mean quantity required—so great a proportion of the rain-fall being in the form of snow during the winter month, and not reaching the stream until March.

In the three years ending October 30th, 1872, the course of the flow was such that we can compare the two systems by years, beginning with November 1st. The average yearly flow being 28 inches, the monthly supply will be 2.33 inches. Under the first system, that of a supplementary reservoir, there would have been stored from November 1st, 1869 to July 1st, 1870, 36.4 inches; by November 1st, this would have been reduced to 9.7 inches by drawing off 28 inches yearly demand. The discharge being stopped then, there would have been, by July 1st, 1871, 28.5 inches, which would have been reduced by November 1st, to 4 inches. By July 1st, 1872, there would have been stored 24.5 inches, and by November 1st, the resorvoir would again have been emptied. If there had been a constant draught of 2.33 inches per month, the greatest amount stored at any one time would have been 19.3 inches, only two-thirds of the amount under the other system.

In these amounts no allowance for evaporation and absorption in the reservoir has been made. The amount of rain-fall lost from these causes in the stream running naturally varied from 11 to nearly 24 inches per annum. In a reservoir presenting a large surface of water, the amount

in addition lost from evaporation would be considerable. Observations on evaporation from water surfaces have been made at the receiving reservoir in New York since 1864.* The mean annual evaporation for six years was 39.21 inches, 81 per cent. of the rainfall. Observations were made on the West Branch from June, 1866, to October, 1870. The apparatus consisted of a box 4 feet square and 3 feet deep, sunk in the earth in an exposed situation, and filled with water. The mean annual evaporation was found to be 24.15 inches, while from a similar box at the receiving reservoir the amount observed was 35.6 inches. Part of the difference may be due to difference of latitude, elevation and distance from the sea-coast; but more is due, I think, to differences in the mode of measurement. My observations were made twice a day, and any apparent discrepancies were checked and corrected at once. The other observations were taken only once a month, the difference between rain-fall and the reading of the gauge on the box being taken as the evaporation. I consider the method followed on the West Branch the more reliable.

The greatest evaporation per year there noted was 27.7 inches. If we allow 28 inches, and assume the area of water surface to be 7 per cent. of the water-shed, the loss from evaporation will be equivalent to 2 inches of rain-fall. The loss from leakage and absorption ought not to be assumed as less than double this amount, making the total loss in the reservoir equal to 6 inches of rain-fall.

The maximum loss from other sources, as stated above, was about 24 inches, the average being 18.5 inches. We will certainly be on the safe side, therefore, in assuming the loss of rain-fall at 30 inches per annum. With a mean rain-fall of 48 inches, we would have an available supply of 18 inches per annum. If, for safety, it was preferred to take the mean rain-fall of the years 1871 and 1872, the available yield would be 12½ inches. This would seem, however, to be carrying precautionary measures to an extreme point, for the actual available yield for these two years was (allowing 6 inches for reservoir loss) 19.30 inches per annum.

For purposes of comparison, there is given in Table III. the result of computations made by Mr. J. P. Kirkwood on the flow of the total water-shed of the Croton river, from which the supply to New York is drawn.†

* These were made with an apparatus designed by Mr. B. S. Church, the Assistant Engineer of the Croton Aqueduct, and described in the report of the Croton Aqueduct Board for 1864. The results were published in the annual reports of the Board until 1870.

† This table is compiled from one given by Mr. Joseph P. Davis, in his report on "Additional Water Supply for Boston," January, 1873.

This water-shed includes that of the West Branch, but is 16 times as great, comprising 335 square miles. The proportion of flat and cultivated land is much greater. The rain-fall is estimated from gaugings taken at three points, one of which was outside the basin. The flow is estimated from the daily observations of the heights on the overflow of the dam (260 feet long), and from the depth of water flowing through the aqueduct. These give a rough approximation to the quantity of water which fell and which flowed off. As might be expected, from the character of the two water-sheds and from the different methods of measurement, the flow from the whole valley bears a smaller proportion than that from the West Branch to the rain-fall.

TABLE III.

RAIN-FALL AND FLOW OF CROTON WATER-SHED, AS COMPUTED BY MR. KIRKWOOD.

YEAR.	RAINFALL. Inches.	FLOW. Inches.	PERCENTAGE Flowing off.	YEAR.	RAINFALL. Inches.	FLOW. Inches.	PERCENTAGE Flowing off.
1864.....	40.80	14.892	36.5	1867...	48.88	30.649	62.7
1865.....	52.43	20.563	39.2	1868...	60.87	34.804	57.2
1866.....	48.57	25.208	51.9	1869...	52.21	25.553	48.9
Mean....	.....	.....	.....	.....	50.62	35.278	49.93

COMPARISON WITH WEST BRANCH GAUGINGS FOR SAME PERIODS.

	CROTON VALLEY.			WEST BRANCH.		
	Rain.	Flow.	Per cent.	Rain.	Flow.	Per cent.
May 1 to Dec. 31, 1867.....	38.59	20.415	52.2	40.43	23.822	58.9
Jan. 1 to July 30, 1868.....	26.91	18.980	70.5	24.02	22.027	91.7
Oct. 1 to Dec. 31, 1869.....	19.99	7.077	35.4	17.85	13.735	77.0

For 1868 and 1869, Mr. Kirkwood's estimate of rain-fall is very much greater than my measurement. In comparing the two sets of observations, it must be borne in mind that the flow at the Croton Dam has already been subjected to the losses from evaporation and the like, which I have considered above.

As before stated, it is evident, from an inspection of the tables and diagrams, that no single gauging will afford any criterion of the quantity of water which may be collected from a stream, nor will the gaugings of any one month be more reliable. I find, however, in analyzing the results of the West Branch gaugings, and also Mr. Kirkwood's results from the whole water-shed of the Croton, that a very close approximation to the proportion of flow to rain-fall for any one year is obtained by gaugings continued through the months of May and June in that year. These are the months in which the flow from any water-shed might be expected to approach most nearly to that which is due to the topographical and geological character of the region. In this latitude, by May 1st, the snow has disappeared, and the excess of flow produced by its melting has passed off. Up to the end of June the heat has not been so excessive as to dry up the ground. Table IV shows the results of this classification.

TABLE IV.

COMPARISON OF

GAUGINGS FOR MAY AND JUNE, WITH THOSE OF THE WHOLE YEAR.

WEST BRANCH WATER SHED.

May and June.	Rain.	Flow.	Per cent.	Year ending.	Rain.	Flow.	Per cent.
1867.....	12.14	8.787	72.381	May 1, 1868.....	51.13	36.169	70.739
1868.....	13.32	9.659	72.513	July 1, 1868.....	52.31	37.071	70.866
1870.....	4.36	3.148	72.200	November 1, 1870....	49.02	37.689	76.886
1871.....	9.18	4.413	48.072	November 1, 1871....	46.00	22.385	48.664
1872	7.69	3.463	45.032	November 1, 1872....	39.08	24.082	61.623
Mean of five years.....			63.118	Mean of four years' gaugings.....			62.916

CROTON RIVER WATER SHED.

1864.....	8.27	2.714	32.8	December 31, 1864...	40.80	14.892	36.5
1865.....	13.09	4.911	37.5	November 1, 1865....	51.62	21.292	41.3
1866.....	8.68	4.442	51.2	November 1, 1866 ...	48.35	22.092	45.7
1867.....	12.16	7.195	59.2	November 1, 1867....	51.17	32.979	64.4
1868.....	15.94	8.816	55.3	November 1, 1868....	57.95	33.428	57.7
1869.....	7.71	3.732	48.4	November 1, 1869....	48.83	26.123	53.5
Mean of six years.....			48.30				49.93

With the exception of the results for 1872, the percentage flowing off in May and June is within 5 per cent. of that for the year. The remarkable agreement of these results would seem to indicate that where, for any reason, gaugings cannot be continued for the whole year, they might be made during the months of May and June, and each year's gauging would give nearly the proportion of flow to rain-fall for that year, and the average of several years' gaugings during these months would give very closely the average proportion per year.

The great differences existing in the results for different years show the importance of continuing gaugings for a long time. Where water works are constructed and in operation, the cost of making measurements of rain-fall and flow is very slight, and the knowledge obtained from a systematic continuation of them is of the greatest value when, as so often occurs, the first works constructed for the supply of water to a town begin to prove insufficient. In the designing of new works the want of information on this point is much felt, and it should be urged upon every engineer having charge of water-works to keep a full record of gaugings, and transmit the same to the Society yearly.

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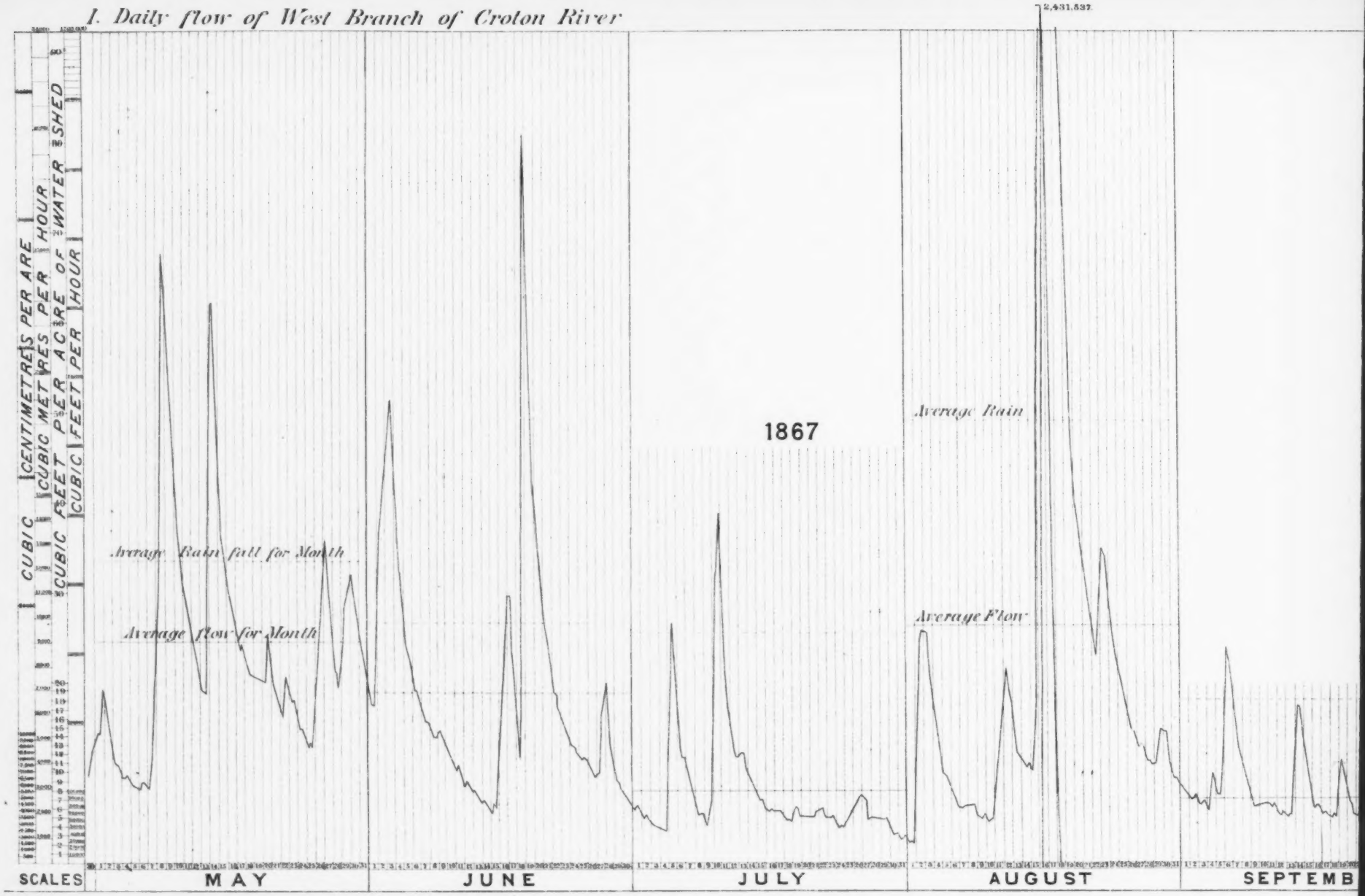
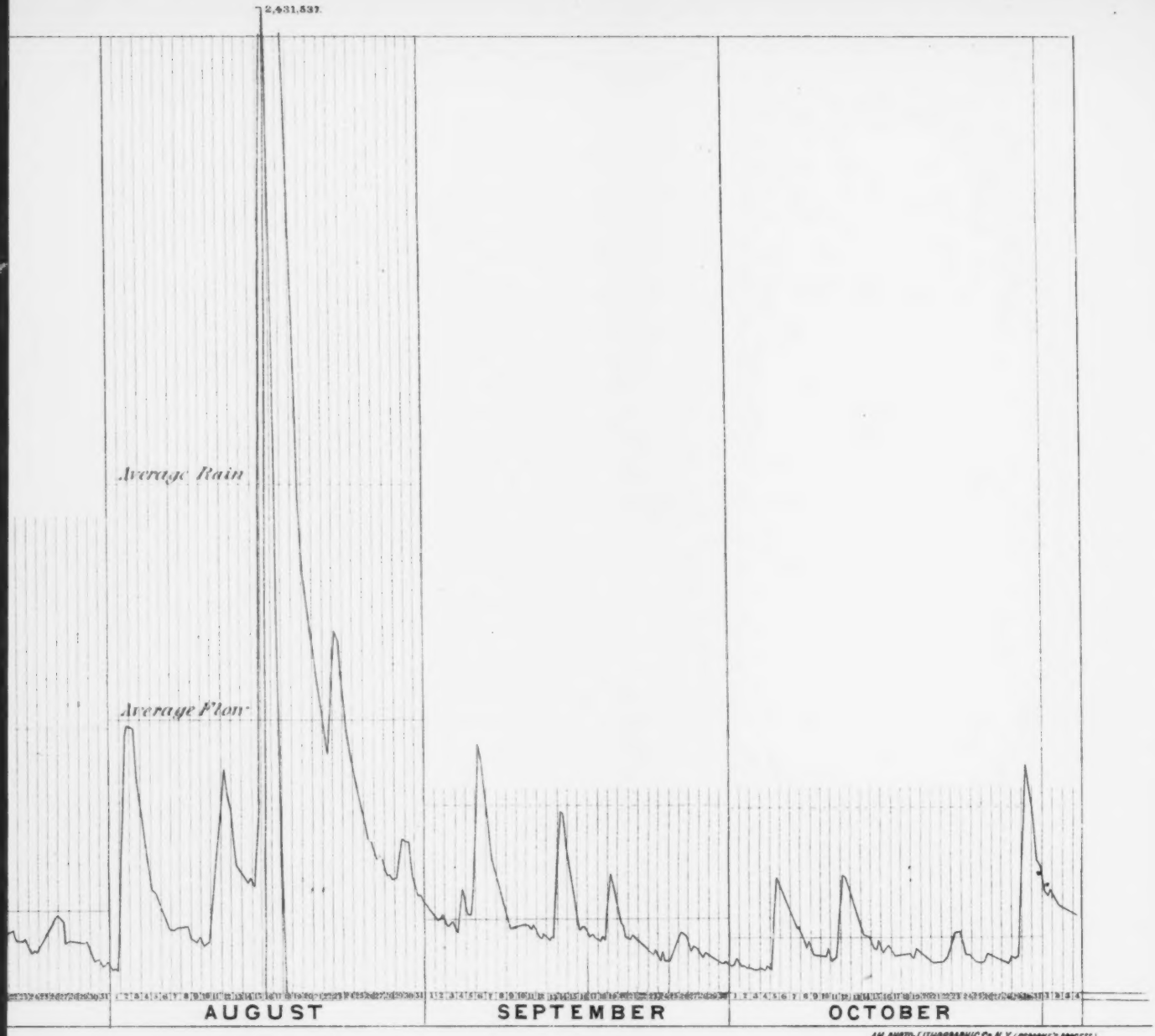
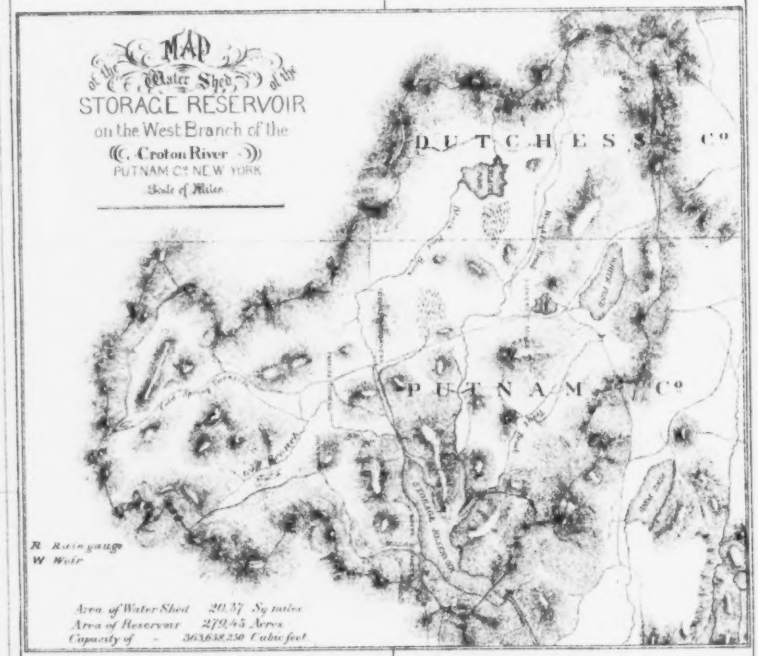

I. Daily flow of West Branch of Croton River
2,431.837
CUBIC CENTIMETRES PER ARE
CUBIC METRES PER HOUR
CUBIC FEET PER ACRE OF WATER SHED
CUBIC FEET PER HOUR
1867
Average Rain
Average Rain fall for Month
Average Flow
Average Flow for Month
SCALES
MAY
JUNE
JULY
AUGUST
SEPTEMB



AM. PHOTO-LITHOGRAPHIC Co. N. Y. (OSBORNE'S PROCESS.)

II. Daily flow of West Branch of Croton River



1870

CUBIC CENTIMETRES PER ARE
CUBIC METRES PER HOUR
CUBIC FEET PER ACRE OF WATER SHED
CUBIC FEET PER HOUR

Average flow

Average Rain

Average Rainfall for Eight Years

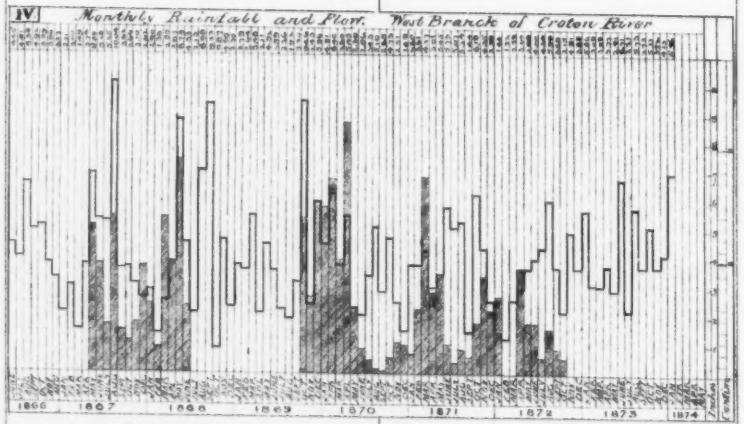
Average Rain and Flow

SCALES

APRIL | MAY | JUNE | JULY

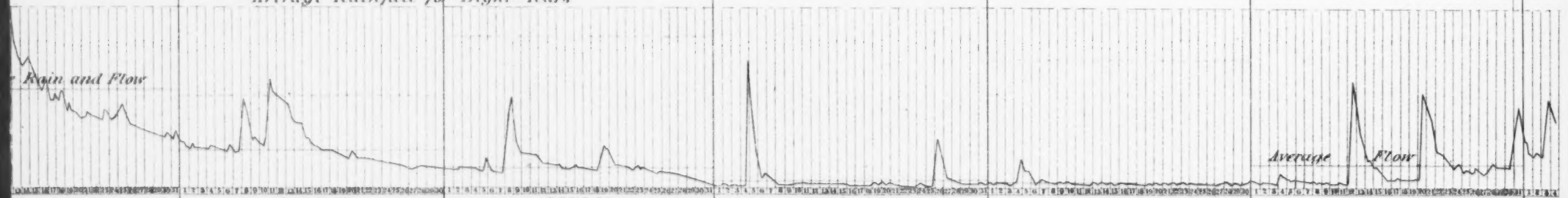
of Croton River

1870



Average Rainfall for Eight Years

Average Rain



MAY JUNE JULY AUGUST SEPTEMBER OCTOBER

AM. PHOTO-LITHOGRAPHIC Co N.Y. (OSBORNE'S PROCESS)

LXXXVIII.

ON THE FORM, WEIGHT, MANUFACTURE AND LIFE OF RAILS.

A Report by ASHBEL WELSH, C. E., M. N. FORNEY, M. E., O. CHANUTE, C. E., and I. M. ST. JOHN, C. E., Members of the Society.

ADOPTED JUNE 10, 1874.

The members of the Committee (or such of them as have been able to meet together) appointed to investigate and report upon the forms, sizes, manufacture, tests, endurance and breakage of rails, and also the comparative economy of steel and iron rails, have given attention to the subject, and issued circular interrogatories* to obtain the results of American experience. They have not, however, as yet received a sufficient number of responses to warrant a final report, or to give full numerical results, on a subject so extensive, so important, and in some respects so unsettled. This they hope to do, at the next Annual Meeting of the Society, and accordingly urgently repeat their requests for answers to their interrogatories.

There are some points, however, on which experience has enabled them to form opinions so decided, that they now submit them to the Society. Members can compare these with facts within their own knowledge; and the Committee, on hearing from them, will, if such facts require it, modify their opinions in the final report.

Before entering upon the particular points on which the Committee is to report, it will be proper to present some general considerations bearing upon all of them.

A rail has two principal functions or classes of functions; acting as a beam or girder, to carry the heaviest weights between the most distant *actual* supports, without straining the metal beyond the limits of its elasticity: and to distribute the weight bearing on one point of the rail among the adjacent supports. The first part of this function requires strength, the second stiffness. The second class of functions or duties of the rail is to resist crushing, and to endure the wear. The first of those requires hardness and breadth of surface, the second hardness

* A copy of which can be had upon application.

and area of section to wear off, or consumable area. We shall consider the rail as divided into two parts, the consumable and the residual ; the latter including what is left of the head after the consumed part is worn off, and all below it. The endurance of the rail, if of good quality, depends on the consumable. The residual must have the requisite strength and stiffness after the other is worn off. These should be carefully distinguished. What is good for one is often bad for the other.

In England, where the supports are far apart, and the weight on a driver is sometimes as much as 16,000 pounds, and where the enormously loaded wheels of the 4-wheeled cars are several times as far apart as the supports, strength and stiffness are the controlling considerations. But in this country, where supports and wheels are both close together, much less strength and stiffness are required, and too much of the latter quality actually diminishes the longevity of the rail. After reaching a safe moderate surplus of strength (not only for new, but for well-worn rails), our whole attention should be directed to increase the wearing endurance.

So in a wooden bridge, a comparatively moderate amount of timber is sufficient for strength to support the load if quiescent, till the timber rots or is burned. But our early wooden bridges, strong enough to carry the loads, and durable with very little traffic, wore out in a few months under a heavy traffic, from insufficiency of the small surfaces of contact to bear the vibrations without rapid wearing. They did not break down, but wore out. Hence wooden bridges were condemned, not because the material was bad, but because there was not enough of it.

Rules derived from the experience of one country should be adopted only with great caution in another. As the controlling consideration in England is strength, and here wear, English rules thoughtlessly adopted have seriously misled us.

The material (its kind, hardness, strength and other qualities), form size and connections should be adapted to each other, to the ties, ballast and road-bed, to the grades and curvature, to the weight on the size and surface, elasticity and contour of the wheels rolling over it, to the speed of the trains, to the climate, to the amount of traffic, to the cost, to the rate of interest, and to the facility of raising money on the credit of the company. One pattern or size or kind of rail cannot be said to be right or wrong, without considering how it is to be fastened, and how it is adapted to the other things mentioned.

To judge of the best means of preventing the destruction of rails, it is necessary to keep in mind the modes and causes of their destruction. The principal modes in which rails become unfit for service are, break-

ing, crumbling, laminating, splitting, splintering, scaling, mashing, wearing down the top, wearing off the running side of the head, breaking off the projecting part of the head when worn down thin, hammering down the ends, and otherwise disfiguring the ends. We take no notice of those injuries, such as breaking the base, which result from obsolete and vicious appliances, such as chairs.

The principal proximate causes of these injuries in this country are brittle, crumbly, lamellar, splintering, scaly, soft, unequal at the same level, imperfectly welded, weak, or permanently strained metal; insufficient wearing surfaces of the head in proportion to the weight on it and the hardness and elasticity of the metal, unevenness from working or wearing down soft spots, so making shocks, uneven or inelastic foundation, ties too far apart or rotten, extreme cold, supposed to render the metal more brittle, and rendering the road bed uneven and inelastic, bad joints, very flat wheels and accidental collisions or blows.

The principal remote causes are, metal from bad stock, such as cold short, red short, cinder, &c.; insufficiently worked, burnt, imperfectly welded, too much or too little carbonized, too high or too low tempered, permanently strained in cooling of unequal hardness or otherwise badly manufactured; punching, head too small; stem and base so heavy as to make the rail too rigid; under side of head too steep or narrow, or curving to hold the fish-plate—fish-plate too short, narrow, soft or weak; uneven surface or alignment of track; road-bed uneven, clayey or wet; ballast too shallow or too coarse, or of unequal depth, stone ballast on clay without intervening stratum of sand or gravel, or very fine broken stone; unequal settling of embankment or earth under the ballast, ties rotten, too far apart, too thin, short, soft or badly bedded; base too narrow for the surface and hardness of the ties; steep grades, sharp curves; high speeds, heavy weights, bad springs, wheels small or with inelastic faces; sliding the wheels; wheels worn into grooved faces, dirt on track, numerous stops and starts; changes of climate, and of course heavy traffic, and the enormous amount of unnecessary dead weight of cars continually dragged back and forth.

Some of these proximate causes act by the abrasion of sliding wheels, as on steep grades, in stopping or reducing speed, on curves where one wheel must travel farther than the other during the same revolution, and from one part of the same conical wheel having a different circumference from another part traveling over the same rail—and by traction. In many cases the too great weight on a small area of rail surface crushes the metal and destroys its cohesion, and thus abrasion easily wipes it off.

In traction the minute grain, fibres or particles of the top film of the rail are alternately pulled one way by the driver, then the other way by the other wheels, till finally loosened and rubbed off.

Rails, as well as machinery, on each system of railroads ought to be in uniform classes, each class proportioned to the traffic and other controlling circumstances. In violation of this obvious principle, it has been common for our railroad companies to pick up such rails as could be had at the lowest price, when the purchase could be postponed no longer, without much regard to form or adaptation, and with no knowledge, and scarcely a thought, of the quality. It would be just as wise to buy a horse on the same principle, without considering whether he is wanted for draft, saddle or carriage, or whether he is really worth $500, or not so many cents. When railroads are made to sell, this plan seems to answer the builders' purpose, whatever may be said of its honesty. Probably $20,000,000 or $30,000,000 a year are wasted in this country by such purchases, and many lives and limbs sacrificed.

Most forms of rail originated with, or were modified by the rail makers, who naturally adopted forms easiest to make, rather than those of greatest durability.

Each material should be put in such form as best to avoid the dangers to which that material is subject. This will be considered in speaking of patterns of rails. As material and manufacture improve, economical forms can be adopted, which would formerly have been impossible in this country, or at least inadmissible. The best form for steel or good iron could not have been used with iron imperfectly welded.

The first point we are to consider is "the best form for standard rail sections for the railroads of this country." The form should be adapted to the material, to the fastenings and joints, to the foundation, to the traffic and to other things, some of which we shall consider as we go on. The form should be such, after securing a reasonable surplus of strength, that where there is most wear, there should be most metal to sustain it. The parts where there is no wear, should be as light within practical limits, as experience has shown to be safe.

Experience shows that all the wear, on a reasonable good rail, except in some vicious arrangements, is on the head, and the force of the blows given by passing wheels is expended and exhausted nearly all on the head. As much of the metal as possible should be put there, where it is most wanted. A steel or good iron rail with good foundations and fastenings and splices, and the weight on the wheels properly proportioned to the hardness of the rail top, finally becomes unfit for service only by

wearing down the top; except on curves, and where the wheels have too much or too little play between the rails, by abrasion of the flange on the running side of the head. With such conditions the duration of the rail is measured by the amount of wear the head can endure, or the sectional area of the metal that can be worn away without weakening the rail too much—that is the consumable part of the rail.

The head should be broad on the top to give as much wearing surface as possible. It has been erroneously supposed that it does no good to make the head more than about 2½ inches wide, because the coning of the wheels prevents more than that from being used. But an examination of rail surfaces nearly 3 inches wide, shows that in fact they are evenly worn all over—forming nearly a plane surface slightly inclined downwards, towards the running side; that the top of the head is actually widened by the spread of the metal on the outside, and that the part so pressed out is also run upon and utilized. Whatever may be the contour of the face of the wheel at first, it wears into such form as to bear all across a wide rail.

Not only is the amount of wearing surface in a broad head greater than in a narrow one, but a square inch of the former endures more than a square inch of the latter. The area of contact between a given wheel and a wide surface averages greater than that between the same wheel and a narrow one. The more the weight is diffused and the less the pressure per square inch, the greater the tonnage each square inch will carry. Injury increases faster than weight on a given space; after reaching a certain point, many times as fast. Probably 100 tons, with a weight of 12,000 pounds on a wheel would injure an iron rail as much as many thousands of tons with 3,000 pounds on the same wheel. This will be referred to again, when we speak of the relative weights of load and rail.

So a broad head gives more surface for adhesion without injury. A rail with a head 1¼ inches wide and 2½ deep would, with very light machinery, last longer than if the head was 2½ inches wide and 1¼ deep, but with machinery as it is, not half as long.

The breadth of the head and the depth necessary for strength being ascertained by the ordinary calculation for beams, and by experience under actual or similar circumstances, the additional depth, to give consumable metal should depend on the amount of traffic. Supposing the rails to be steel or iron that will only give way by wearing out, let R represent the cost per mile of the residuary part of the rail, C the cost of the metal the traffic is expected to consume in each year, L the loss, including in-

convenience and incidental expenses on each renewal, besides the cost of the metal to replace that consumed, T the interval between renewals, and a the rate of accumulated interest for that time. Then $c\,T$ will be the consumable part of the rail, and

$$R + c\,T + \frac{L + c\,T}{a} = V^*;$$

the value of a rail that will last forever = present value of the cost of the rails and their renewals forever, traffic being constant. Of course T, which determines the consumable depth of head, should be such as to make the above value V, the smallest possible.

Suppose the residuary part of a mile of steel rails costs $7,000, and its renewal $4,000, then if $C = 850$ per annum, T should be about 30 years, so that $c\,T = 1{,}500$; if $C = 100$, T should be about 20, so that $c\,T = 1{,}800$; if $C = 200$, T should be about 16; if $C = 400$, T should be about 14; if $C = 600$, T should be about 12; if $C = 800$, T should be about 10, and if $C = 1{,}000$, T should be about 8, so that in this extreme case the consumable part of the head should cost $8,000, or more than all the rest together, and the head would be 2½ inches or 2¾ deep.

In forming the original section of the head, some regard should be had to the form given to it by its subsequent use. But if a small addition of metal will prevent it from wearing down to the ultimate form for a long time, that addition should be given instead of beginning with the partly worn form. The top corners of the head should not be much rounded, for that diminishes both the width of the top and side wearing surfaces, and even if rounded at first, it becomes nearly angular on the outside by use.

It is common to give the top sectional curvature of the head a radius of 6 inches. It is better to give it about twice that radius, for in a short time the surface approximates a plane, and in hammering down the too great central convexity a permanent strain is created tending to split the head. This too great convexity also tends to wear a groove on the faces of the wheels.

The under side of the head should be as broad as possible, so as to have the greatest practicable bearing on the top of the fish-plate. Experience shows that joints often work by the indentation of the under side of the head, and the battering down of the top of the fish-plate, especially when the latter is short, and so the rail ends are bent down and finally destroyed. This indentation is often imperceptible, except

* For the principle of this formula see a paper on the comparative economy of Steel and Iron rails by the Chairman of the Committee, herewith presented.

from the sound and tremor. The outer corners of the under side of the head should therefore not be rounded as usual, but sharp. This also gives greater wearing surface to the side of the head. There is no objection, except in looks, to a sharp salient edge, as there would be if the angle were reëntering. All these considerations show that the section of the head should approach the angular form instead of the oval.

The under side of the head and the top of the base near the stem should, of course, be absolutely plane surfaces, so that the fish-plate shall fit and be in entire contact, its whole thickness, both when new and when drawn nearer the stem after wear. The inclination of the top of the base and bottom of the head, where in contact with the fish-plate, should be about four horizontal to one vertical, or about 14° from horizontal. If much steeper than that, even if considerably flatter than the angle of friction, the fish-plate has a tendency to work outwards with the vibrations (just as a loose object works down an almost level tremulous plane) becomes loose, though perhaps not perceptibly, so as to wear the shoulders, the rail begins to wear and to suffer strains, and it weakens and finally breaks the bolts. If much flatter, it weakens the head and base, and does not allow sufficient wedge movement to tighten up when the shoulders wear.

A very large proportion of the fish-joints in use are only imperfectly successful, because the shoulders are too steep, or too narrow or not plane surfaces, or the plates are too short, too narrow on the edges, or of too soft metal. They should be of steel.

The short and therefore inefficient fish-plates formerly used in this country followed English precedent. With their plan of setting the rails on chairs the joints must, of course, be between chairs, and the ends of the rails kept from bending down, only by the fish-plates. They must, therefore, be very short or they would bend. There is no such reason in our case. Their plates are 14 inches; ours should be, and often are, twice that length.

Again, in England, steel fish-bars are condemned, because with their steep shoulders, short plates, rigid rails, and great strains, the steel breaks, while iron bends. There are no such severe strains in our joints. The Chairman of the Committee has, within the last 7 years, used probably over 100,000 steel fish-plates, and so far as he can hear not one has ever broken.

The pernicious influence of the rules derived from the experience of one country on the practice of another, when the circumstances that induced those rules do not exist, is strikingly shown by these examples.

From such cases we can understand how engineers and managers, whose professional education has been exclusively or mainly European, or from European books of practice, labor under extreme disadvantages in this country.

It would be well to make the fish-plate longer and thinner towards the ends, so that in case of a vertical bend of the line of rails near the joint, an angle should not be formed at the end of the plate, but a vertical curve. The tops and bottoms of the plate should retain their thicknesses so as to be stiff sidewise.

Where the fish-plate is used, the angles between the sides of the stem and the under side of the head and the top of the base are necessarily sharper than seem on other accounts to be desirable. With good iron, and especially steel, this objection is very small. But with imperfectly welded or very inferior iron the objection becomes serious; and it would perhaps be better in such case not to use the fish-plate, but to fill in the reëntering angles and make the joint fastening at the base. With worthless iron, good forms cannot be had.

The stem has generally been made unnecessarily thick. Experience with many thousand tons under a heavy traffic during the last 8 years has shown that a thickness of one-half inch of iron and seven-sixteenths of steel does not perceptibly compress nor bend sideways, nor with a good fish-joint, or any other good joint, give out in any other way.

The stem should not be made thick for the following reasons: being near the neutral axis it adds little to the strength; it wastes material and unnecessarily increases cost. If the unnecessary metal in the stem of the rail in diagram No. 17 had been put into the head, it would have increased the duration 33 per cent. It is more difficult to put poor metal into a slender stem. It sometimes makes the rail too rigid and so increases the wear of the head and the liability to break. With a given width of head, the thinner the stem, the wider the bearing of the fish-plate under the head. When the head becomes elongated by hammering and crushing its surface, a thin stem yields more than a thick one, and so does not transmit so great a longitudinal strain to the base.

The base should be broad so as not to cut into the cross tie. 4 inches does very well for good white oak ties, covering full 30 per cent. of the ground under the rail; but is insufficient with chestnut or other ties of soft wood unless, like cypress, it is very elastic. With soft wood ties on a road with heavy traffic they should cover 40 per cent. of the ground under the rail, or the base should be 4½ inches wide, or even more. This width, formerly impossible, or very difficult without great thickness or

permanent strain in cooling, can now be attained with good metal without a great increase of weight. One-eight inch thickness at the edge is sufficient to transmit the pressure to the tie without risk of breaking. Great breadth of base has been advocated, on the ground that it increased the metal at the point farthest from the neutral axis, and so increased the strength. On the other hand it has been denied that the outer edge increases strength at all, because it curls up as the rail bends. The last view is probably right, but it is of little consequence, compared with other considerations, which is right. The most important relations of the breadth of base are not with the strength of the rail, or with its height, but with the amount and hardness of the timber under it. Experience once showed that rails 7 inches high, with about half that breadth of base did not upset. The preponderance of weight on the running side of the head is more than enough to counterbalance the centrifugal force on curves.*

The base should be thin, because when the consumable part of a steel or good iron rail is nearly worn off, at which time the question of strength becomes prominent, even the thinnest base is heavier than the residuary head. Not only so, but part of what is left of the head being compressed, the other part is permanently strained, and hence is liable to break over a tie. Metal, for strength, should, therefore, be put into the head rather than the base. A rail generally breaks, not from weakness of the base but from injury to the head. A thin base saves metal and cost. Only good metal can be put into it. It prevents too great inflexibility. It is notorious that very rigid rails are more liable to break than those moderately flexible. Instead of conforming to the slight irregularities and undulations of our American foundations, a rigid rail can neither carry the weight between points so far apart as those it is liable to rest upon, nor bend down to some intermediate support.

The relative weights of the many conflicting considerations in favor of high and of low rails can be known only by experience. A high rail is more rigid, and therefore the head wears out faster, and, under some circumstances, the rail is more liable to break. The heads of the rails 7 inches high, already referred to, did not wear near as long as the same sized heads on lower rails. On the other hand, a low rail does not give sufficient width to the fish-plate, brings the flange of the wheel in contact with ice and dirt, and, if very low, is not stiff enough. Experience

* Mr. Robert H. Sayre, Chief Engineer of the Lehigh Valley R. R., has adopted a pattern shown by diagram No. 6, in which the base and height are both 4½ inches.

seems to have sanctioned a height of about 4½ inches for rails of 60 pounds weight per yard, over 4 inches for those between 50 and 60 pounds, and 3½ inches for those under 50 pounds. Many, who once strongly objected to these heights now adopt them.

The reasons for light base and stem, and for putting all the metal possible in the head, suggest the question, why not make all head? In other words, why not use a rail, say 2½ inches square, which would weigh about 63 pounds to the yard? Besides the inconveniences of so low a rail and the greater risk of poor metal, it would be too flexible, if not at first, certainly when partially worn, while the rail, with stem and base, would be stiff enough when much more worn; and with so narrow a bearing, the square bar would cut into the tie, and the connections would be unsatisfactory.

In the patterns of rails hereinafter recommended, the head contains from 45 to 50 per cent. of all the metal. We prefer 50 per cent. to anything less, except for a road but little used. On such a road the rail must be nearly as strong as on any other, but wearing endurance is not required.

Experience has shown that the thickness commonly adopted for stem and base is from one to three-sixteenths inch more than is necessary, adding a nearly useless weight of about 8 per cent. to the weight and cost of the rail; or, perhaps, $30,000,000 to the cost of the rails now laid in this country.

As rails often break from punching the stems and slotting the bases, and the fracture begins in those parts, we may fairly be asked, if this danger would not be diminished by making them heavier. Doubtless it would; but that would be a very expensive way of diminishing the danger, while it can easily and cheaply be prevented by proper fishing with a long stiff fish-plate extending far past the last hole in the direction of the middle of the rail, with broad plane surfaces of contact between rail and plate, with flat slopes of shoulders, in which case the bolts and nuts do not work loose, the plate bears the head and the weight on it, and carries it to a distance from the ends of the rail, and the rest of the rail, near the end, has nothing to do. With other than fish-joints, we admit, there is some force in this objection.

Bolt holes should be drilled, not punched, through the stem. The base should not be notched at all, but the rail held from traveling longitudinally by straps connecting fish-bolts and ties.

The Committee, as now advised, is of opinion that the patterns shown in the plates annexed, Nos. 1 and 2 for iron, 3 for steel, and 4 for

either first-rate iron or steel, the weights being respectively 62, 50, 53 and 56 pounds to the yard, all formed according to the foregoing principles, are substantially of the right form and proportions, unless the machinery is extravagantly heavy. Many thousands of tons of the 1st, 2d and 4th of these patterns, and many hundred of the 3d, have stood the test of several years' experience under a heavy traffic, especially on the different roads between Baltimore and New York. The corners, at the bottom and outside of the head, should not be so much as in the first four of these patterns, but the sides and bottom of the head should be brought very nearly to a sharp edge, as represented by the dotted lines. This would give greater wearing surface on the sides of the head, and greater bearing surface for the fish-plate, and would not increase the weight one-fourth of a pound per yard, or less than 0.4 ton per mile.*

Mr. O. Chanute, Chief Engineer, Erie Railway, and member of the committee, proposes to adopt the form No. 5 for the Erie Railway.

The Committee cannot see any definite relation between the proper weight of a rail and the weight of a wheel rolling over it. Of course the rail, both when new and when fully worn, must be strong enough to carry the weight between the most distant actual supports, and to distribute that weight among adjacent supports. As this depends on the residual part of the rail, there is, within certain limits, a sort of rule of thumb relation between the weight of that part and the weight on a wheel. But if patterns or materials differ widely, even that relation fails. That most important to consider is, between the weight on a wheel and the hardness, elasticity and breadth of the metal in the head. The proper relation of the consumable part of the rail is not with the weight on a wheel, but with the amount and mode of the traffic. The relations of breadth of base are with weight on rail and amount and hardness of timber under it.

As the weights of different parts of the rail should be adapted to different purposes, and so follow different rules, the parts should not be increased proportionably, but each according to the duty it is to perform. Nearly as strong a rail is required for a light as for a heavy traffic, with the same machinery, but it would be folly to put on as heavy a head

* For some particulars of experience with rails of the first four of these patterns, the Committee refers to the annexed Memoir contributed by their Chairman.

Mr. Sayre has slightly beveled the upper part of the head in the rail shown in No. 6. This makes weight and cost about one per cent. less than if the sides were carried vertically up, and consumable area, and therefore endurance, three per cent. less. His object is to save the flanges of the wheels. The Committee is not prepared to give an opinion on this feature.

for a light traffic road as for a heavy traffic. A poor rail gives out by coming to pieces, a good one by wearing down the head; neither is cured by general increase of weight. If our ties were always sound and solid and their surfaces in the same plane, the lightest rails in common use would be sufficient for strength and stiffness. But this is not always the condition, and the frost makes it worse by heaving the road-bed unequally.

Supposing the actual supports to be 5 feet apart or under, the rails represented in the diagrams are sufficient to carry the loads. According to more than 100 trials, made under the direction of the Chairman of the Committee several years ago, with rails on supports 15 feet apart, the deflections of these rails were as follows: of the 62 pounds iron rail, 1.25 inches, with 4,400 pounds centre load; of the 50 pounds iron rail, 1.57 inches, with 3,500 pounds, and of the steel, shown by diagram No. 4, 1.44 inches, with 3,800 pounds. The calculated elongation of the bottom film of the rail was in each case one fourteen-hundredth. In every instance the deflections continued to increase in proportion to the weights beyond those here given. The great distance between supports, of course, caused the twists and unsteadiness and lateral bends of the rail to show a worse result than if the supports had been nearer together. With actual supports, 5 feet apart, the rails should safely bear centre loads as follows: the 62 pounds rail, 13,200 pounds, with deflection of 0.14 inches; the 50 pounds rail, 10,500 pounds, with deflection of 0.17 inches, and the 56 pound steel rail, 11,400 pounds, with deflection of 0.16 inches. Occasional weights much heavier could be carried without sensible injury. For the partially worn rails, the stiffness being less, the distance between actual supports would in most cases be diminished by enabling the rail to bend down to an intermediate support. But a more important evidence that these rails are strong enough is, that after a heavy traffic for several years they do not break, whether new or with the head fully worn. As the actual bearings and other data for an imperfect or frost-heaved track are unknown, not calculation, but experince must decide what strength is sufficient.

Our rails do not give out from want of strength, but from injury to the head, not always perceptible to the eye. After a certain amount of such injury, additional strength postpones but does not prevent the destruction of the rail. It is better to prevent destruction by a little more, and especially by a little better metal in the head, than merely to postpone it by much more metal below the head.

The really important inquiry is, what is the relation between the

size and elasticity of, and the weight on a wheel, and the width, hardness and elasticity of the top of the rail.

Experiments made by Mr. Chanute show that a newly turned driving wheel, 5 feet in diameter, bears upon a rail surface only about one-fourth square inch. If, therefore, the weight on such a wheel is 10,000 pounds, the static pressure on the rail surface in contact with it is about 40,000 pounds per square inch, and the dynamic pressure is still more. If the metal is so soft as to crush under this weight, as most wrought-iron does, the top of the rail must not only be disintegrated so as to be easily wiped off by abrasion, but it must be expanded and elongated, a permanent longitudinal strain must take place, tending to tear apart the lower part of the head and all below it, and if it is not soon worn out, ultimate breakage must ensue. Increase in weight, at best only postpones this breakage for a short time. Many rail tops are actually elongated one-fourth inch.

As the crushing and consequent expansion of the metal extends but a short distance below the surface, any additional metal put in to counteract the strain should be put, not into all parts of the rail proportionably, but into the head, where it answers other important purposes.

A weight of 5,000 pounds, on a wheel 30 inches in diameter, probably causes very much more than half the pressure per square inch that is caused by 10,000 pounds on a 5-foot wheel. And the same weight on the same sized wheel, with the wheel or rail or both highly elastic, gives much greater surface of contact, and therefore less pressure per square inch, than if both were inelastic.

When the weight on a square inch area of rail surface reaches a certain point, a small addition to it increases the damage many-fold. Increase of width of head tends somewhat to remedy this; but a general increase in proportion to the weight of the wheel would do very little good. We, therefore, consider that no rules fixing the relative weights of load and rail should be adopted, except within very moderate limits for rails of the same pattern, kind and quality. This would be so local and limited as to be of no general use. As an increase of weight in the stem and base may cause too great rigidity, and thus diminish endurance both by increasing wear and fracture, any such rule is liable to be not merely useless but mischievous.

The Committee is, as yet, unprepared to go into detail on the manufacture of rails. We remark that the manufacture has so improved in this country, that patterns formerly inadmissible on account of the

difficulty of rolling and risk of defects, and difficulties in cooling, are now readily produced.

A few general principles, already familiar, may be briefly enumerated. The top surface should be very hard, and if possible elastic, and made of such materials and in such manner as to be uniform, especially where heavy machinery is used; the base should be of strong and, if possible, slightly ductile metal, the cooling and straightening so managed as to cause no permanent strain of one part of the rail against another. A judicious increase of 10 or 15 per cent. to the cost of ordinary rails would double their value. But this, rail purchasers will not often pay, and rail makers have no encouragement to put on the additional work.

For testing rails, the drop and the dead weight have often been used too exclusively. They test the rail only as a beam, and the metal only as to brittleness and strength. In England, where strength is the controlling consideration and when the rails are double-headed, the drop may be the best test. But here, where rails do not give out from weakness but from wear, and where a certain amount of flexibility is conducive to the longevity of the rail, and where there is only one head, it is not a sufficient guide, except in comparing different rails of the same kind and pattern. A rail of light head and heavy base will give the best test with the drop, but it will not last so long as a rail with heavy head and light base. The 56 pounds steel rail, diagram No. 4, is not so strong or stiff as the 62 pounds iron rail, diagram No. 1, and will not give so good results with the drop, but it will last 10 times as long. Of course, we would not reject it as one of many tests. Reported tests commonly give us permanent set. What we really want to know is not the amount of set, some weight or blow gives, but how great the weight or blow can be, without any set at all.

Many years ago, the Chairman of the Committee adopted the test of hammering the rail to destruction. Perhaps 40,000 tons were tested (by specimens taken at random) in this way; and the experience is, that the number of blows on any kind and pattern of rail very closely measures the lifetime of that rail as compared with others of the same kind. It fully discloses the defects of the rail, as brittleness, softness, bad welding, lamination, &c. This, however, does not give the relative endurance of iron and steel, as it probably would if each was destroyed only by honest wear. Steel lasts many times as long as iron, not only because harder, stronger, more elastic and homogenous, not coming to pieces, but, perhaps, mainly because as yet the weight on its surface is not great

enough to crush it; whereas the weight on the same surface of iron being too great for the material, crushes and destroys the fibre.

As most tests are related to the mode of manufacture, it is best to defer their consideration till the manufacture is considered. We only remark that the "durometer" is advantageously used to test the hardness. Iron rails give out very often from unequal hardness in the head, which makes it wear down in spots. This instrument can be used to detect any such inequality.

"The endurance of rails" is so dependent on their kind, quality and form, and their foundations and splicing, and on the grades, curves, &c., of the road, and on the speed of the trains, and size and weight on wheels, no definite average numerical results can be given without a much fuller response from the different railroads than we have yet obtained. We shall hereafter give some particular results of experience on this point. The duration of a rail is as indefinite as the value of a horse or the size of a piece of chalk. Some of the early rails were in use 25 or 30 years, while new and larger rails laid in the same track during part of the same time, gave way in 2 or 3 years. Perhaps the light hammering of early times actually increased their endurance. With heavy machinery and high speeds a rail might carry 1,000,000 tons; with very light machinery and moderate speeds the same rail might carry 10,000,000 or 20,000,000 tons.

Of course the means of increasing endurance besides increasing the sectional area of the consumable part of the rail, are better metal, good road-beds, moderate speeds and moderate weights. As other than engineering considerations often control the weights of machinery, the present excessive weights must be accepted as fixed.

One obvious remedy, though only partial, is to increase the width of the top of the rail, so that the surface of contact of each wheel being greater, the pressure per square inch shall be less. If the weight on a driving wheel is 10,000 pounds, and its area of contact with a rail head 2.1 inches wide, is one-fourth square inch, the pressure will be at the rate of 40,000 pounds per square inch. But if by increasing the width to 2.8 inches the area of contact is increased to one-third square inch, the pressure will be only 30,000 pounds per square inch. The damage done to the rail in the former case must be many times as much as in the latter.

Another obvious means of promoting endurance is to ballast the road with stone broken very much smaller than is customary in this country, and cover 30 per cent. or more of the span with ties about 7 inches thick, and not less than 8 feet long. The length of ties does not depend mainly

on gauge, but on weight and bed. In some soils 9 feet lengths are required not to slide. Good ballast, in most situations, can be put on for one track for $2,000 per mile. If the rails will last 4 years with poor ballast, they will probably last 5—with good. Experience has sometimes shown much more than that difference.* If the cost and inconvenience of a renewal is $4,000, the present value of renewals each 4 years forever is $\frac{4000}{0.317} = \$12{,}620$, and each 5 years, $\frac{4000}{0.411} = \$9{,}732$; (with money at 7 per cent.) The difference, $2,888, is nearly 50 per cent. more than the cost of ballast. Besides, the maintenance of the road-bed so ballasted is next to nothing, while that of the unballasted or badly ballasted is very considerable.

The last point on which we are called upon to to report is "the breakage of rails." We shall not dwell on such well known causes as ties too far apart, rotten or uneven, want of good ballast, road-bed yielding unequally under the weight, depressions on the top of the rail, especially at the ends, lateral angles at the joints, or sharp curves approximating angles, punching, slotting and bad joints, nor shall we refer to obsolete appliances, such as chairs, &c. Nor shall we do more than refer to the too great rigidity of rail by which, as before pointed out, their liability to break is increased.

We call attention again to the elongation of the top of the rail by too great weight on a wheel in proportion to the bearing surface, and the hardness of the metal, by which a permanent strain is created on the rest of the rail, perhaps gradually snapping the extended fibres or somehow weakening the metal, resulting in ultimate fracture. We also repeat that this is partially remedied by greater width of the top of the rail.

But why do rails break in winter especially?

1st. Because the material is, as most of the Committee think has been proved, really more brittle in very cold weather. According to experiments reported by Sandberg, specimens requiring a drop to fall 39 feet to break them, with the thermometer at 84 , were broken with the same drop falling 11 feet, with the thermometer at 10 . Different kinds of iron and steel vary very much in this respect. Steel, though more elastic, does not wire-draw like iron. Iron does not wire-draw so much when

* Many years ago the Chairman of the Committee ballasted a mile as follows: the loam at sub grade was rolled and leveled till even slightly rounded and uniformly hard, then a thin stratum of fine gravel put on to prevent the loam from rising between the broken stones, then a stratum of ordinary ballast 6 inches thick, then very fine ballast 9 inches thick, in two courses. Each stratum and course was thoroughly rolled before the next was put on. For 10 years it is not supposed a dollar was spent on this mile, except to watch it. Rails and ties are supposed to have lasted 40 per cent. longer than in other places.

cold. But it may be questioned whether the limit of elasticity in iron or steel is lowered by cold. If it is not, then cold alone would not endanger the rail, but inordinate strain must be an essential condition of fracture.

2d. The road-bed heaves irregularly, causing the rail to bear on few points far apart.

3d. The road-bed being frozen hard becomes inelastic; and also high points are not lowered to the level of other supports by the weight.

4th. The top of the head being elongated, and a permanent longitudinal strain thus created coöperates with the other causes in producing fracture.

5th. Insufficient allowance for contraction by cold may cause a longitudinal tension acting against the fish-bolts.

From a report on this subject by Mr. Chanute, it appears that on a Western railroad there had been no breakage for 10 years after the rails were laid, then 200 or 300 in one winter, and then over 5,000 the next winter, showing that by use the rails became more breakable.

Our last duty is to give our views of the comparative value of steel and iron rails. While steel rails as we get them are tolerably uniform in quality, iron varies so much that no comparison can be made except of particular qualities or of averages of qualities widely different. We can as yet do little more than give the results of our own experience. In so doing we shall not only compare steel and iron, but also the effects of some different circumstances on the duration of both.

It seems probable that the best iron, if homogeneous and the head of uniform hardness, so as to wear off evenly like steel, would, with machinery of moderate weight, wear a third or even half as long as steel. The Chairman has found that his 62 pounds iron rail after carrying about 14,000,000 tons gross load, has worn off only about 25 per cent. more than the steel rails on the same track and under the same circumstances. Probably it will not wear so well when the top crust is worn through. But owing to want of homogeneousness and uniformity the iron scales, splinters, laminates or somehow disintegrates or mashes in spots before it wears out.

In 1864 some steel rails were laid down at Wetherly, on what is now part of the Lehigh Valley R. R., on a steep grade where there is constant drilling twelve or fifteen hours a day. A section of the only rail yet taken out after nine years and seven months wear is shown in diagram No. 17—the dotted line representing the probable original top. If that was the top, the head is worn down three-tenths inch, and 9.30 pounds of metal

per yard worn off by what is supposed to be equivalent to 40,000,000 tons freight or 100,000,000 tons gross load. The other rails are expected to last two years longer (in all twelve), and so to carry the equivalent of about 50,000,000 tons freight or 125,000,000 tons gross load. If the stem had been thinner the wear would probably have been less. Iron rails in the same situation averaged four months, so that the steel seems to be thirty-six times as durable as the iron.

The 62 pounds iron rails before mentioned may carry 12,000,000 tons of coal with the slow speeds of the Belvidere R. R. Rails in other places of fair ordinary quality carry from 2,500,000 to 6,000,000 tons freight, or 4,000,000 to 15,000,000 tons gross load, according to weights, speeds, grades and other circumstances.

A steel rail may last five or ten times as long as really good iron rails, fifteen or twenty times as long as those that pass for good rails, thirty or forty times as long as the common run, and fifty or even one hundred times as long as many rails made ten years ago, or since imported from the cinder heaps of Great Britain.

From observations made under the direction of the Chairman, of which diagrams No. 6 to 16 are specimens for steel, and No. 18 to 28 for iron, it appears that 1,000,000 tons of freight or its equivalent, equal to 2,500,000 tons gross load, have worn off the steel from 0.14 to 0.62 pounds per yard, averaging 0.30 = 0.47 tons per mile, and have worn off the iron from the 62 pounds rail rolled at Bethlehem from 0.27 to 0.69 pounds per yard, averaging 0.38 = about 0.6 ton per mile. The rails have been laid from four to seven years, most of them six years. It will be seen from the diagrams that in most cases the head is worn off as much on the side as on the top, even on straight lines. This is owing in part to the gauge of foreign cars being two inches narrower than the road.

If the heads of these steel rails can be safely worn down one half inch, and one-fourth inch from each side, those on straight lines and under other favorable circumstances ought to carry 100,000,000 tons of freight, those on the worst curves 16,000,000, the average being 50,000,000, equal to 125,000,000 tons gross load. But for the difference in gauge between the cars and the road, the endurance would probably be much greater. On straight lines, the wear on the top seems to be only one-hundredth inch for 125,000,000 tons of freight, which should require 62,000,000 tons to wear the head down one-half inch.

Probably steel rails can at the present moment be purchased for $9,000 per mile, which last ten times as long as the best iron that can be

had for $6,000 per mile. The commercial value of the iron (measured by price) will then be two-thirds that of the steel, its intrinsic value or capacity for usefulness (measured by endurance), one-tenth that of steel; its economic value will depend on combined capacity and opportunity for usefulness (measured by endurance and traffic), and the interest chargeable against it.

Let I represent the cost of the iron $= 6000$, S that of the steel $= 9000$, L the loss on each renewal of the iron, say 4000; L^1 the loss on each renewal of the steel, say 5000. Let a be the rate of accumulated interest (at 7 per cent.) for the interval between the renewals of the iron, and a^1 that for the intervals between the steel. Then to make the iron and steel equally economical,

$$I + \frac{L}{a} \text{ should equal } S + \frac{L^1}{a^1}.$$

If iron will last 10 years under the traffic on the road to be provided for, and steel 100 years, then the cost of iron and its perpetuation will be $6000 + \frac{4000}{0.99} = 10041$; of steel, $9000 + \frac{5000}{971.8} = 9005$, and the present value of the difference in favor of steel will be 1036. At 12 years it is nearly equal, over 12 years iron is most economical, while under 12, steel is.

But it may be economical to use iron temporarily, while traffic is light, and replace with steel when traffic becomes heavy. Under no circumstances can it be economy to use a very heavy iron rail (say 80 pounds to the yard), for a light steel rail would cost less and last longer. It makes little difference whether a steel rail will last one-half century or forever. The present value of renewals every 50 years forever is only $\frac{5000}{30.19} = 165$, or less than 2 per cent. on the cost of the rails.

The members of the Committee have endeavored to dissect the subjects referred to them, and present the points separately, so that members of the Society and others may give facts or opinions on points within their observation, before the Committee is called upon for a final report.

A MEMOIR ON RAILS.*

By Ashbel Welch, C. E., Member of the Society.

Read June 10, 1874.

During the year 1865 the task presented itself to me of devising or selecting suitable forms of rails for the system of railroads occupying the central part of the State of New Jersey, between Philadelphia and New York, of which I was the executive officer as well as the engineer. I proposed to use three classes of iron rails, one for main lines with heavy traffic, one for branches with light traffic, and one for sidings but little used, and one class of steel rails, of course, for main lines. The result of the consideration then given to the subject was the adoption of principles by which the forms of rails should be determined, nearly identical with those set forth in the Report to which this Memoir is appended, and patterns were adopted conformable thereto.

A 62 pounds iron rail pattern was adopted in 1865, afterwards slightly modified by less rounding of the corners of the head. This pattern, so modified, is shown in the annexed diagram No. 1, height 4½ inches, base 4 inches, stem ½-inch thick, radius of sectional curvature of top 12 inches, inclination of bearing surfaces for both bottom and top of the fish-plate, 14°. Many thousand tons of rails of this pattern, rolled principally by the Bethlehem Iron Co., were used under my own direction, and, so far as I can learn, no American rails have endured better most not so well. Diagrams Nos. 18, 19, 20, 21, 22, 23, 24 and 25, show how they are worn in 6 years, carrying 6,000,000 tons freight, or 15,000,000 tons gross load. After an experience of 8 or 9 years with such rails, under a heavy traffic, with heavy machinery and high speeds, after much enquiry, I have not been able to find that the half-inch stem has ever compressed or bent sideways, or the head or base has cracked off, or suffered in any way from the sharp re-entering angle with the stem, or that the rail has ever given out, except by wearing, crushing, scaling, splintering, or laminating at the top. Substantially this form is now being adopted by neighboring engineers in this country, who at first strongly objected to it. I made one decided mistake in this pattern, by not having the outer bottom corners of the head sharp enough; or, rather, I yielded too much to the feeling against such an unsightly thing as an angular head.

* Appended to the Report of the Committee on the Form, Weight, Manufacture and Life of Rails.

Where chestnut or other soft ties are used, unless the wood is very elastic, the base of this rail should be increased to 4½ inches.

Diagram No. 2 shows the 50 pound iron rail I adopted for branch railroads; height and base each 3½ inches, stem $\frac{1}{2}\frac{5}{6}$ thick, head 2⅜ wide, and 1⅛ deep. More than 30,000 tons of this pattern, rolled by the Allentown Rolling Mill Co., are in use on different roads, and do good service.

Previous to 1867, the steel rails laid in this country, copied from the iron rails then in use, were unnecessarily heavy, and so proportioned that much of the metal did no good. The forms in use for iron rails were calculated not so much to prevent their destruction by wearing out as by coming to pieces. Believing that there was no danger of this latter mode of failure in a steel rail, and that a very thin stem and base would give sufficient strength, I determined to try a pattern in which all the metal possible should be put into the head, and as little as possible anywhere else. This pattern, intended as an experiment to give the principles I had adopted an extreme test, is represented in the diagram No. 3, height and base each 4 inches, head full 2⅜ inches wide and 1¼ deep, radius of sectional curvature of top of head 12 inches, stem $\frac{7}{16}$ thick, base $\frac{7}{8}$ on the edge, and its upper surface rising at an angle of 14° (same angle as under side of head), weight 53 lbs. to the yard.

After considerable negotiation with Naylor & Co., during the early part of the season of 1866, I gave them the order on August 14th, for 200 tons of steel rails of that pattern, to be rolled by John Brown & Co., at Sheffield. After much hesitation, and unsuccessful effort on their part to get the pattern modified, and the lengths reduced below 30 feet, the rails were finally rolled by them without much difficulty, stood an unexpectedly good drop test (tested, as I understood at the time, by Kirkaldy, at London), and were laid in the spring of 1867 at points on the main lines of the railroads above referred to, where it had been found that rails were destroyed most rapidly.

The severe usage to which these rails were subjected, without the slightest injury, soon made it evident that they were strong enough, and several hundred tons more of the same pattern were laid down in 1868 on the main line, between Philadelphia and New York, in places where rails were worn out most rapidly. One point where they were laid was in Bergen Cut, a mile or two from the Hudson river, where the curves are very sharp and the speed high. Probably, 36,000,000 to 38,000,000 tons of gross load per annum have since passed over each track laid with these spindle-shanked rails at that point, equivalent to 9,000,000 to 10,000,000 tons of freight in the six years.

After frequent, including very recent, examinations of the rails of this pattern, I cannot find that one rail has ever broken, although many hundreds of other rails, including some 67 pounds steel, have broken on the same roads since these were laid. The only injuries I can find to the stem or base are—one base cracked, and the bases at three joints on rotten ties bent up for a very few inches. In many places where these rails are laid the road-bed is not a good one. The rails with thinnest stems wear best.

On applying accurate complementary templets to these rails, no stem has been found compressed or bent or deflected from the perpendicular. The templets would show a deviation of less than one-hundreth of an inch. Except the wear on the heads, the rails are apparently as good as when first laid, some 6 and some 7 years ago. The accompanying diagrams, Nos. 6, 8, 10 and 12, show how and how much the heads have worn. On the sharp curves the wear on the top of head of outer rail is three or four times as much as on straight lines, the wear on side of head increasing the whole loss of metal to four or five times as much as on the straight line, except where there is much stopping and starting. As these rails were in all cases laid in the worst places, the wear is much greater than it would average on a whole road. In those places good fair iron rails, as rails were in those days, wear out in a year. On the Belvidere Delaware R.R., where the trains commonly take 750 tons of coal, at a speed of 8 miles per hour (the weight on each driver being under 10,000 pounds), and the foundation is good, the wear on top of rail is sometimes not over one-hundredth inch for 2,000,000 tons of coal or its equivalent; while at Bergen Cut, when the radius of curvature is under 600 feet, the loss of metal from the top and side of the head is full six times as great for an equivalent amount of traffic. In the former, the capacity of this 53 pounds steel rail is probably 80,000,000 tons of coal, or 200,000,000 tons gross load, in the latter 30,000,000 tons gross load.

These rails having behaved so well, I adopted the pattern for the system of roads in my charge; slightly modifying it, however, by putting a little more metal into the base and stem, making the latter $\frac{1}{16}$-inch thick, and increasing the weight to 56 pounds. This is shown in diagram No. 4, and was partly a concession to general opinion (especially to that of President Hinckley, who adopted this pattern for the Philadelphia, Wilmington & Baltimore R.R.), partly to facilitate the manufacture, but especially to be on the safe side. As the original 53 pounds pattern has given just as good results as the other, I would adopt it now in preference, except when the machinery is very heavy, or the ties bad.

Several hundred miles of track laid with steel rails of this modified (56 pounds) pattern, have been in successful use under a heavy traffic, with high speeds, from three to six years; especially on the roads between Baltimore and New York. I cannot find that more than 2 or 3 of these rails out of 7,000 or 8,000 tons laid under my own direction have been broken or removed from the track.

This pattern, regarded at first by all who saw it with surprise and disapprobation, has been growing in favor, and with slight modifications, extensively adopted. Those who have since adopted substantially the same pattern, may have been led to it, without any knowledge of mine, by the same course of reasoning that I followed.

For reasons elsewhere given, the corners on the under side of the head ought to have been less rounded. With ties of soft wood (unless very elastic like cypress), the base should be 4½ inches wide, or even wider.

Misled by a high foreign authority, I at first used too short fish-plates, only 14 inches long, the ends of which slightly indented the shoulders of the rail. I afterwards used fish-plates 28 inches long. The flat slopes adopted for the bearings of the fish-plate prevent any tendency to move outwards causing the bolts to break or the nuts to work loose. I put small leather washers under the nuts, and though tried periodically, they are sometimes not tightened for many months or even for years.

I understand that some railroads are now using an iron rail of the pattern shown in diagram No. 4. If the metal is thoroughly welded, it will do well.

As the foregoing experience with very slender rails differs widely from what was generally expected, I submit it for the information of the Society.

Mr. Horatio Allen.—In listening to the Report and Memoir just read, I have wished very much that we had in this country the good old English custom of saying "hear, hear." Surely, then, many present would have been prompted thus to manifest their approval. Although all may read the Report in print, those who have traveled a long way to attend this Convention must rejoice that so valuable a paper has been pressed upon their attention and memory by one of such experience and ability. It is rarely a body of engineers listens to so much that is valuable, as we have just done.

Mr. Robert L. Cooke.—I understand this is only a partial report, and that the Committee has the matter still under consideration. I move

the adoption of the Report, and that the Committee be continued. (The motion was carried.)

Mr. J. Dutton Steele.—I have been exceedingly interested in the Report, and think the Society has great reason for congratulation that this important topic has been placed in so competent hands.

When the old fish-belly rail in the Liverpool & Manchester R. R. was abandoned, the matter was made the subject of scientific investigation in England, the result of which was the production of a rail pattern nearly like that recommended by this Committee, the adoption of which, however, was prevented by the influence of the iron manufacturers.

About a dozen years ago, a very large U shaped rail, 12 inches across the base and 6 inches high, was in favor in England. The two rails forming the track were tied together, every 8 feet in length, with iron bars, and the ballast was put in so as to sustain them, without ties or other supports. An English stockholder in the railroad of which I had charge at that time, caused to be sent over to America enough of these rails to lay one mile of track, which was carefully done, and the ballast put in. The result was, that at first it was one of the most perfect roads that I ever rode upon; there was hardly a jar or quiver; but in consequence of the lack of elasticity referred to in this Report, the wear was excessive, and in six months the rails had to be taken up.

The double-headed English rail was intended to reverse. If I am rightly informed, one objection to this pattern not mentioned by the Committee is, that the roll of the wheels on the head of the rail granulates or wears it so that fracture takes place immediately upon reversion. I have watched with some interest, as probably subject to similar influences, steel frogs when reversed, and find that they will not then withstand the wear.

Mr. A. D. Briggs.—My own observation has led me to doubt very much whether low temperature is the cause of failure of rails in cold weather. It is a matter to which I have given much attention during the last few years. I have collected considerable information upon the subject, and so far as I am at present informed, *good* rails and *good* iron wheels are not weaker in a low temperature or more likely thereby to break than in a higher temperature. In Massachusetts it has been shown that few, if any, more rails or tires break in cold than in warm weather, from temperature alone. I am assured the failures in cold weather are due instead, to defective rail patterns, the quality of material, or the condition of the road-bed.

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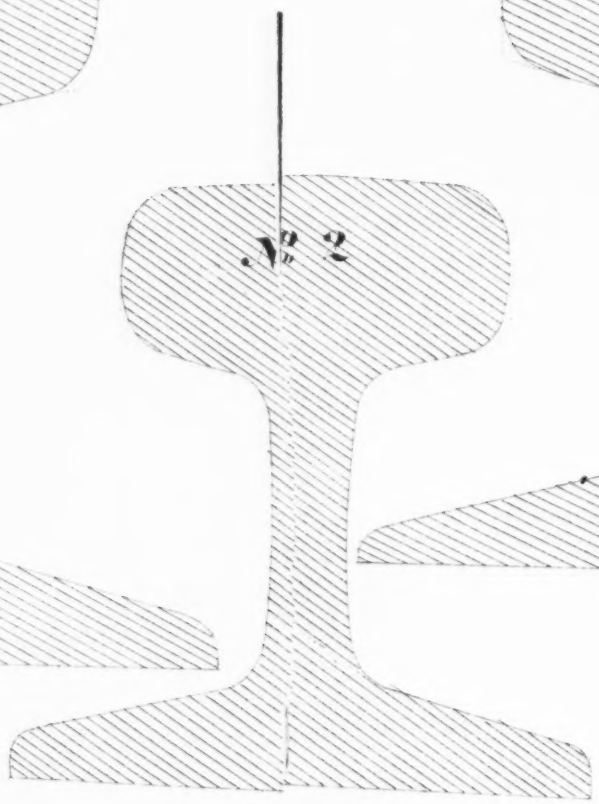

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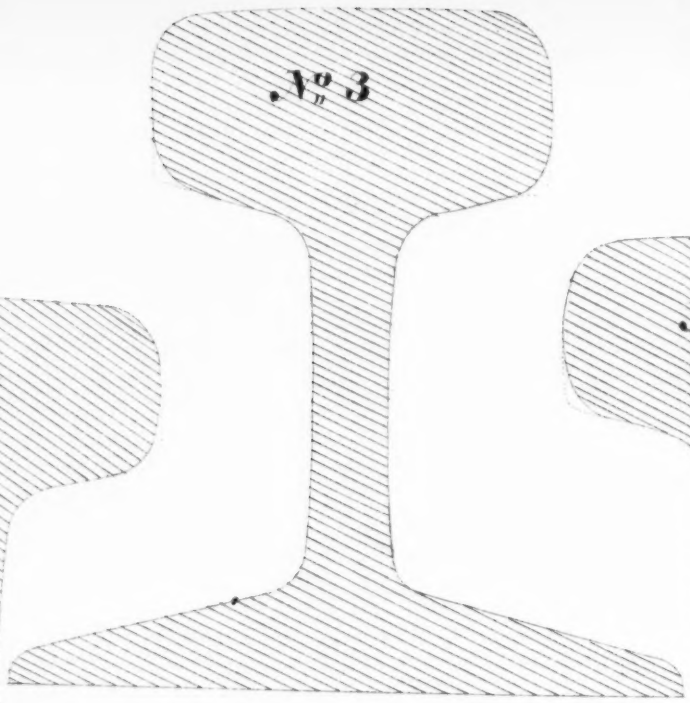

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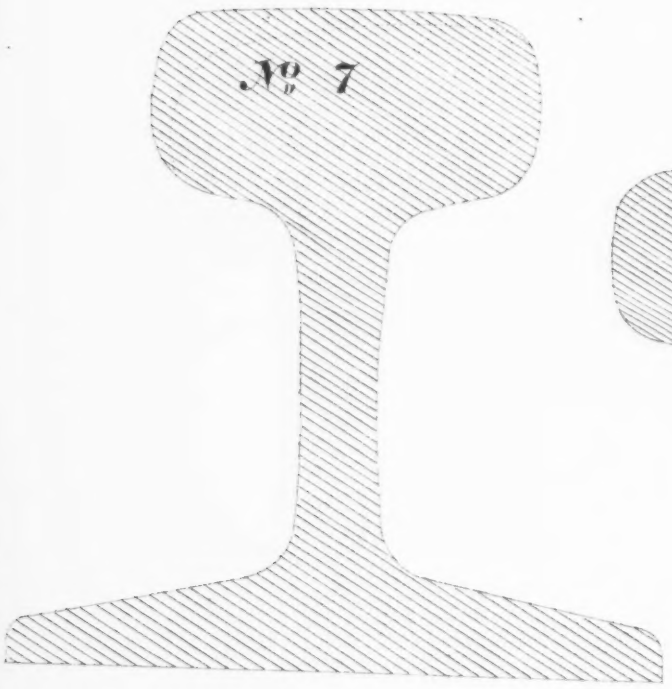

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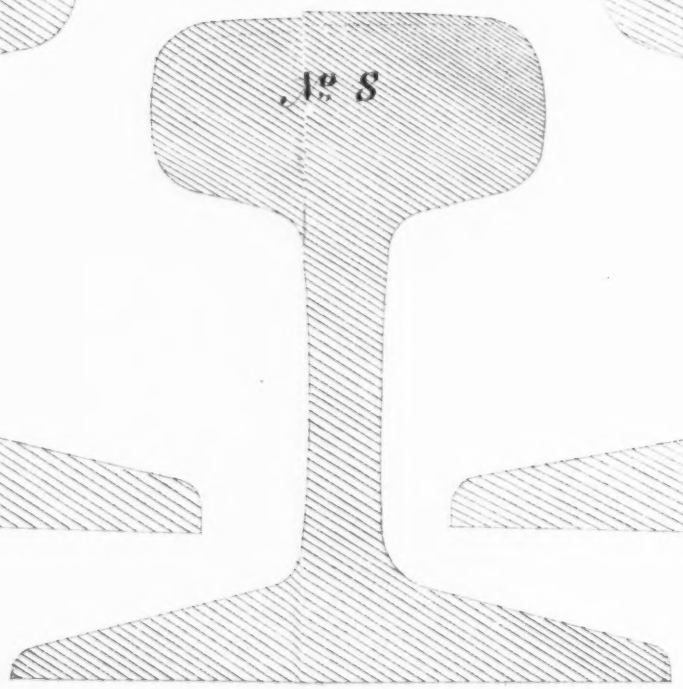

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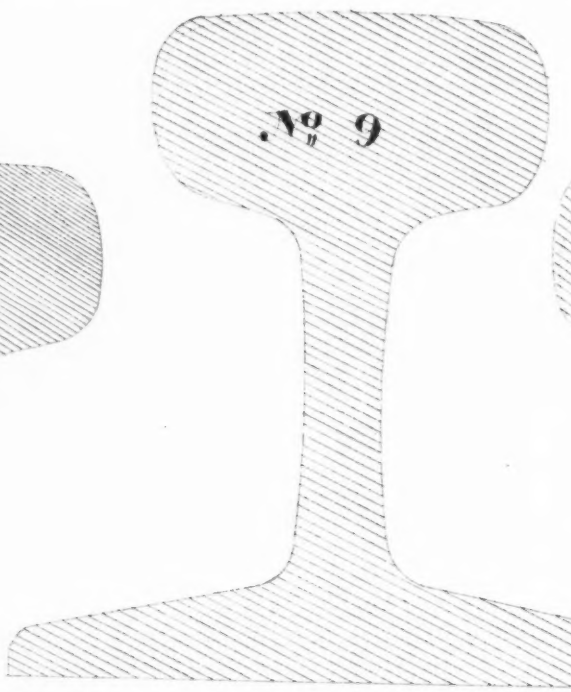

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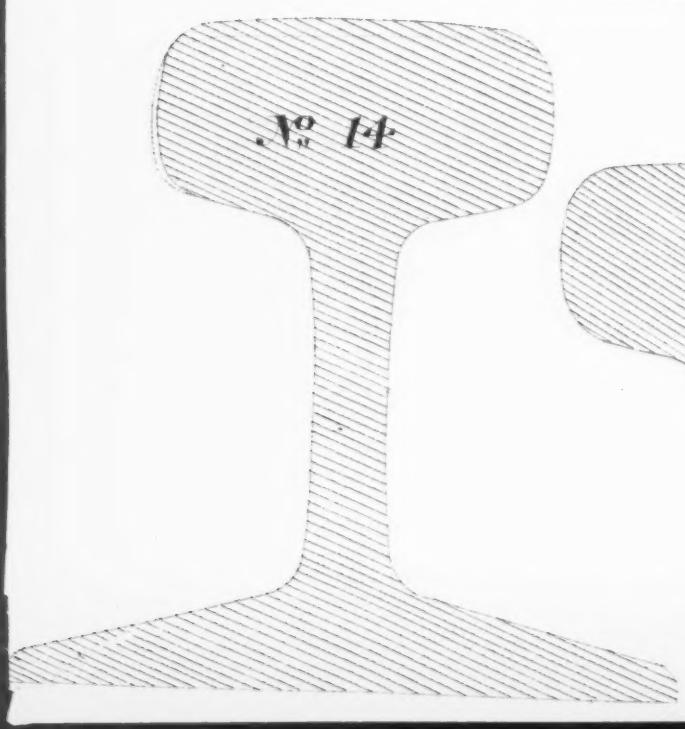

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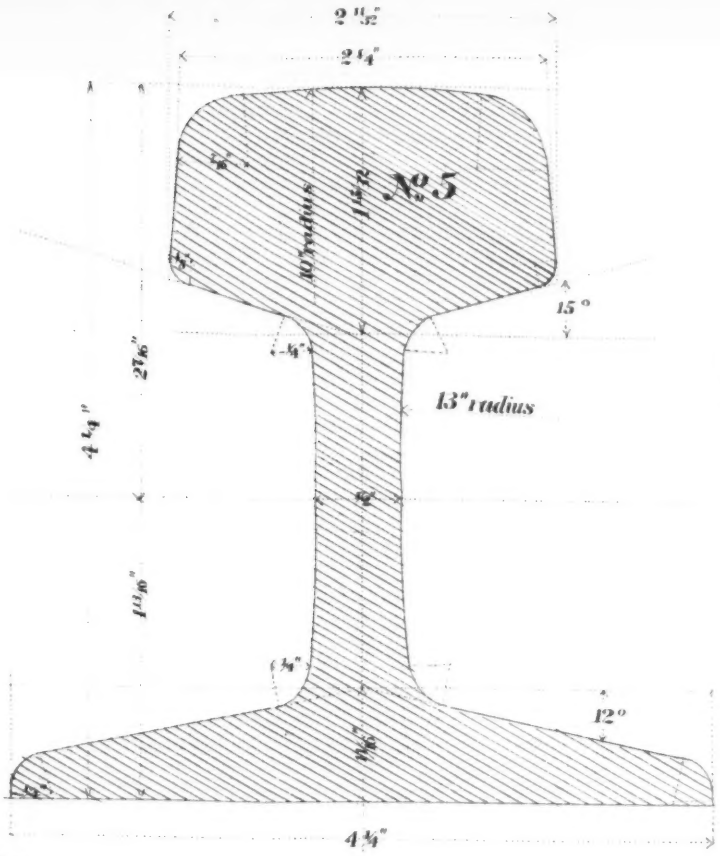
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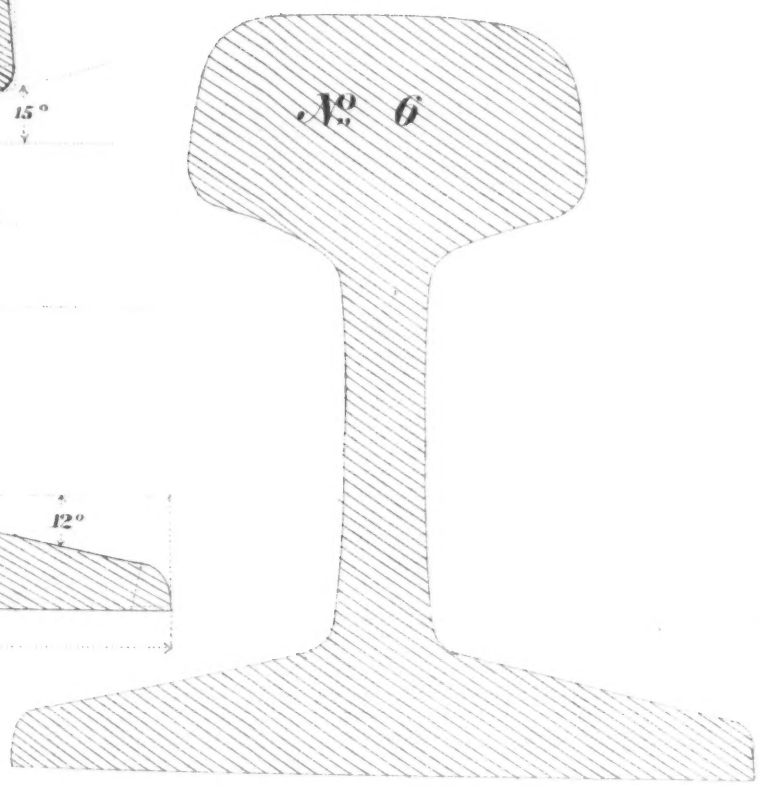
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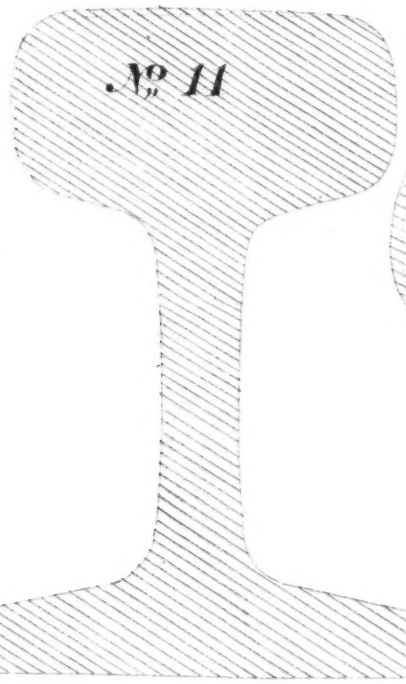
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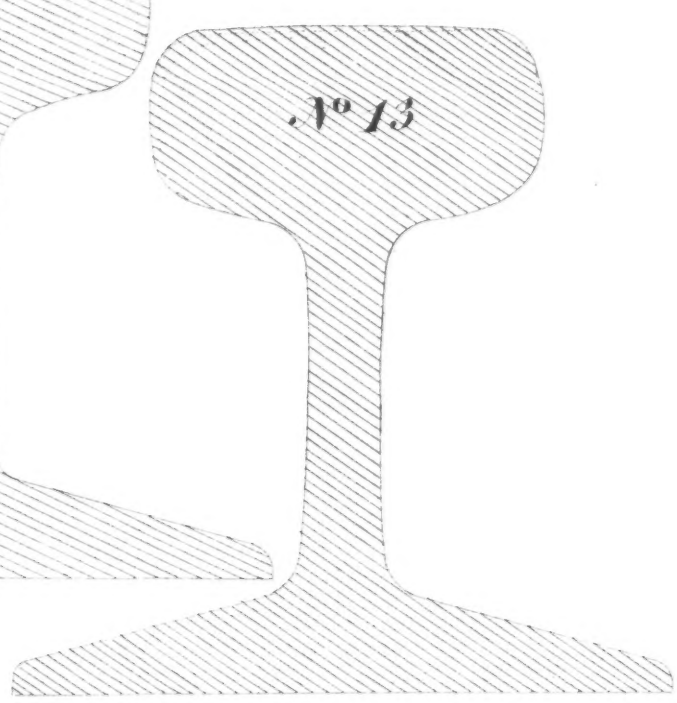
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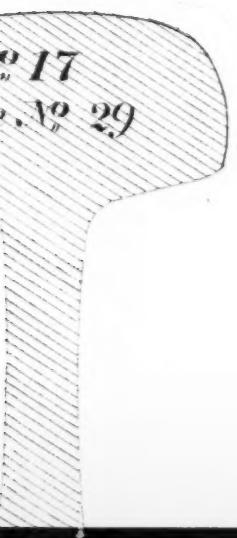
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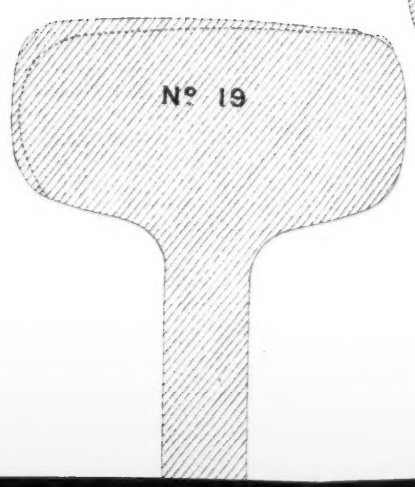
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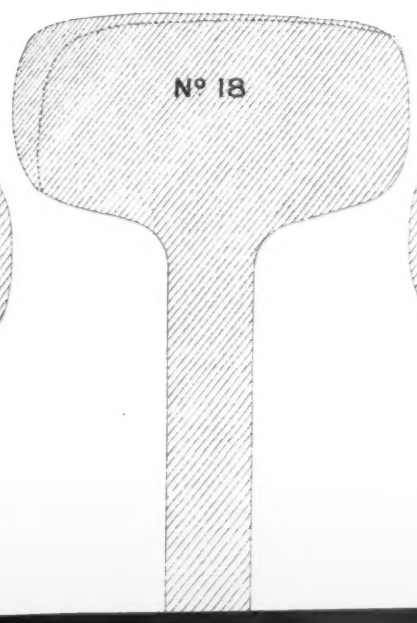
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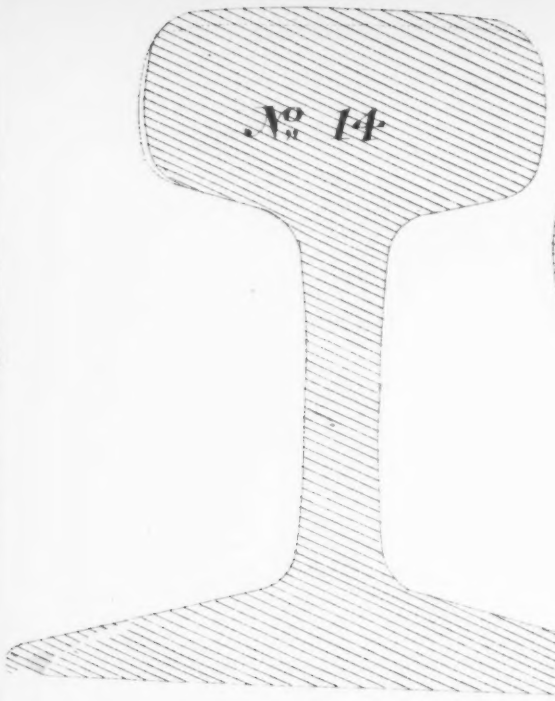
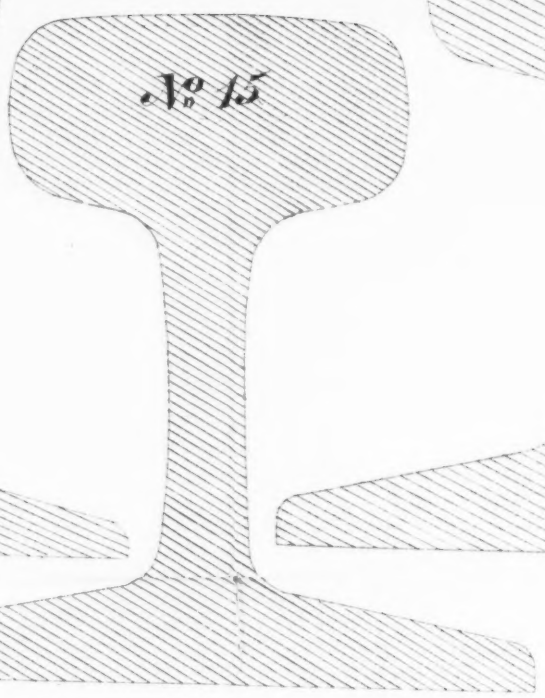
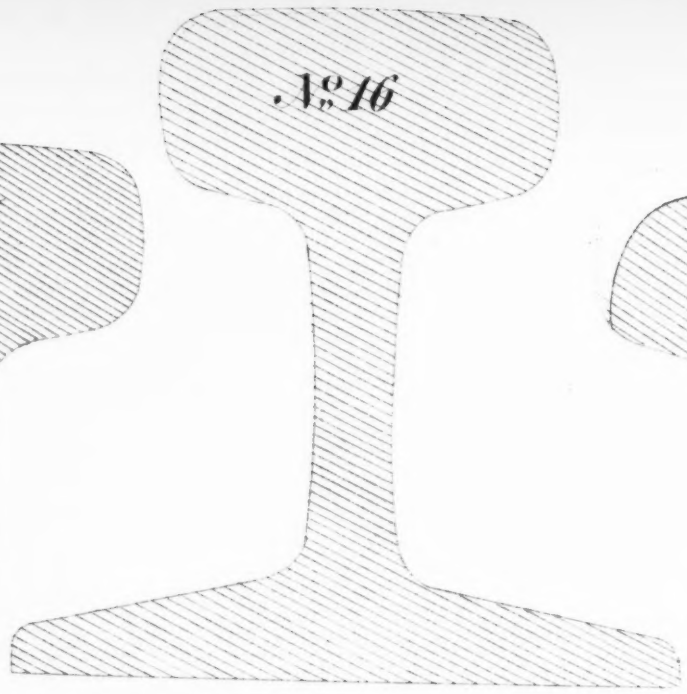
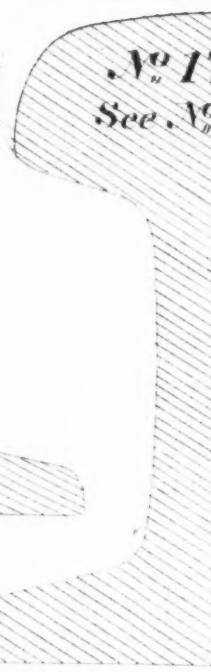
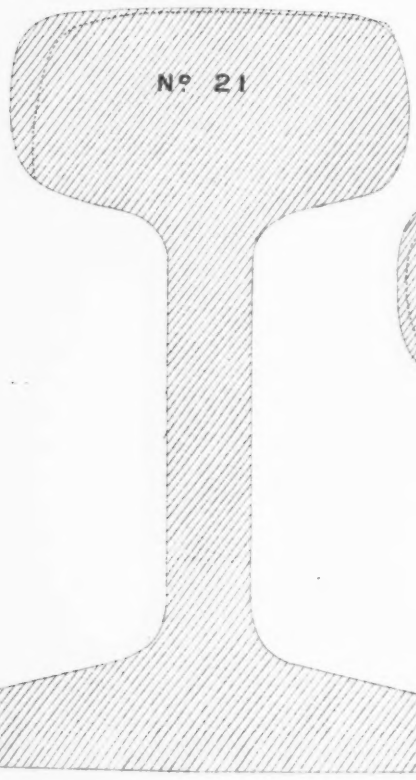
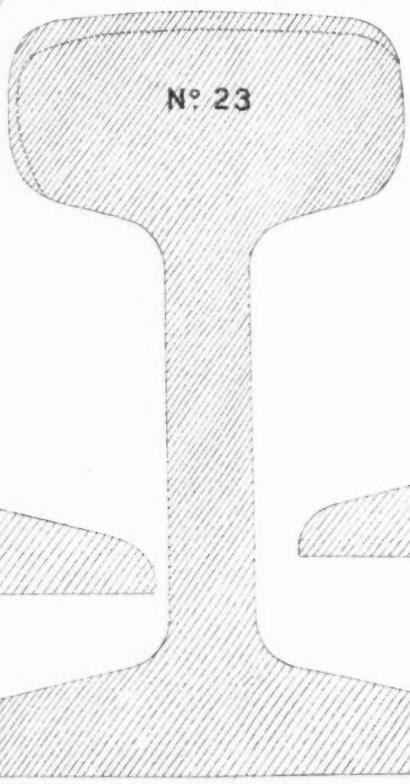


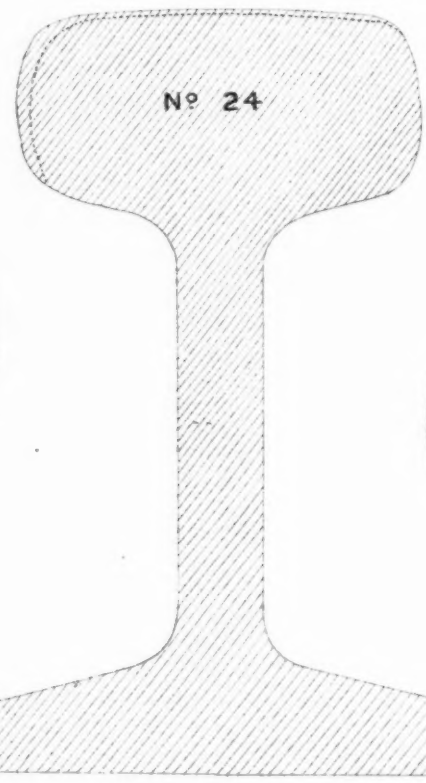


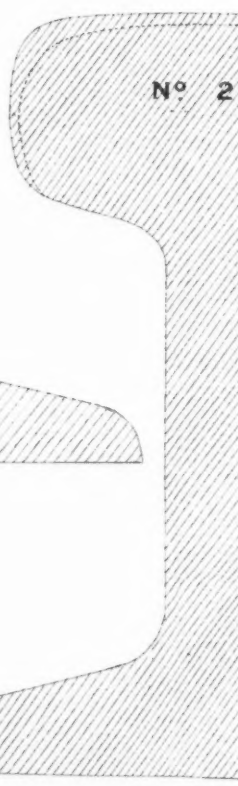


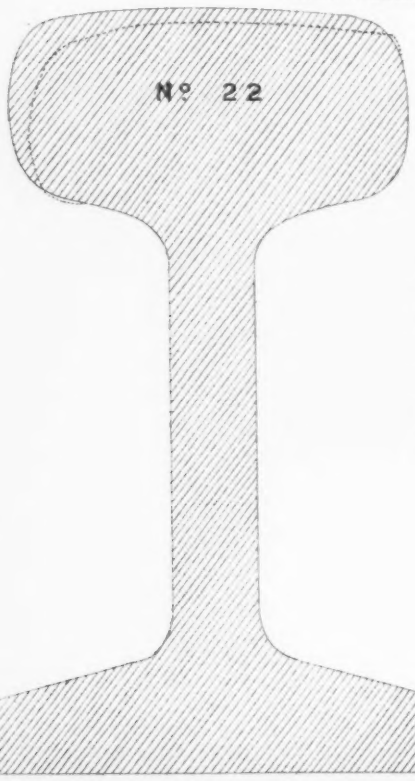








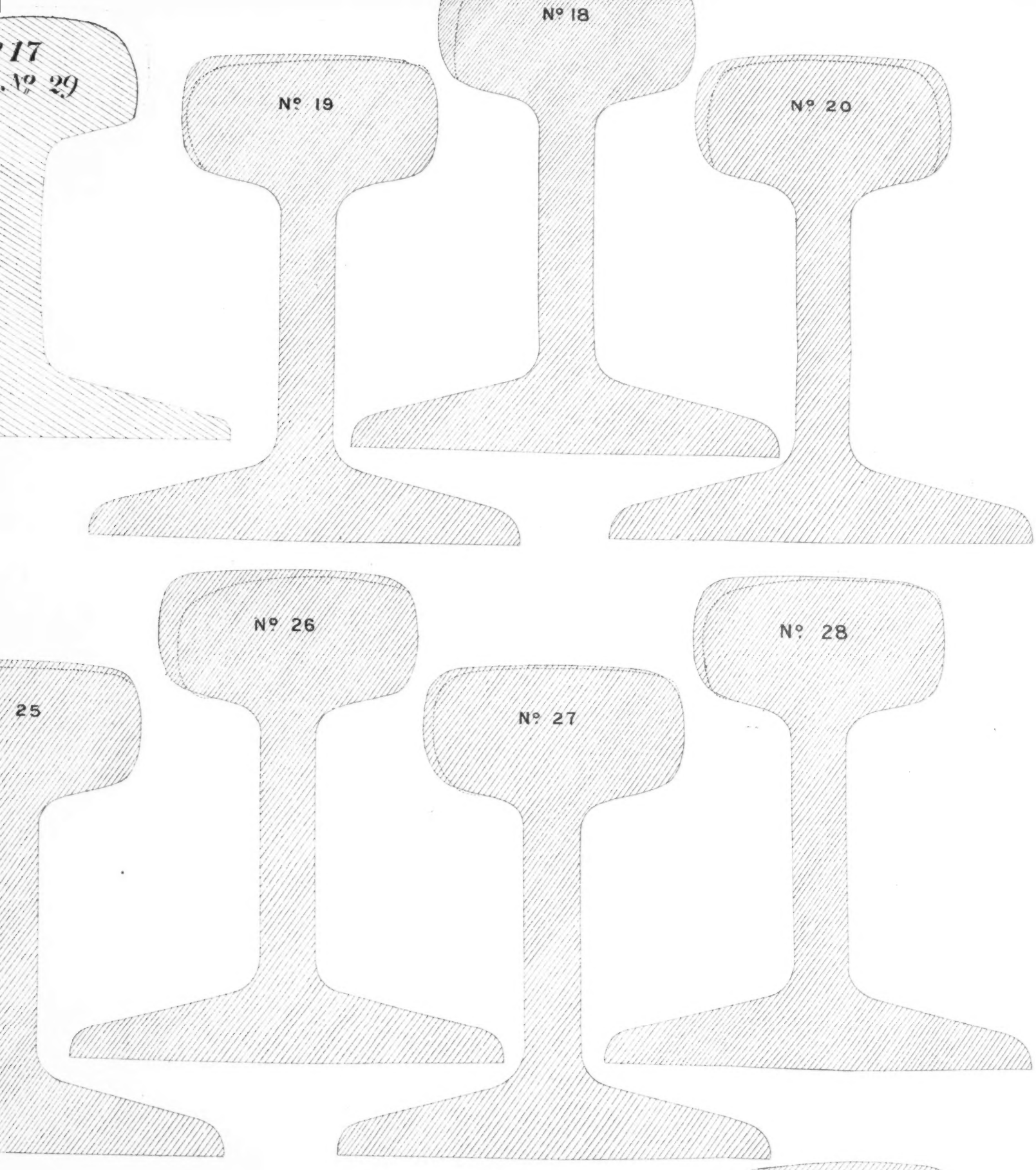




The following references are made to the rail sections which accompany the report of the committee:

No. 1. Iron Rail 62 pounds per yard.
No. 2. Iron Rail 50 pounds per yard.
No. 3. Steel Rail 53 pounds per yard.
No. 4. Steel Rail 56 pounds per yard.
No. 5. Form for steel Rail proposed by Mr. O. Chanute, — pounds per yard.
No. 6. Rail adopted by Mr. Sayre from the Easton & Amboy Railroad, weight 65 pounds per yard. Same pattern for iron and steel.
No. 7. Section of 56 pound rail now in use on outside of curve near front south of bridge over the Passaic River, Center street route, Newark. Made by John Brown & Co., and has been in use four years.
No. 8. Section of 53 pound rail in use on straight line in Jersey City yard. Made by John Brown & Co., and has lain four years.
No. 9. Section of 56 pound rail in use on main line at Newark Meadows. Made by John Brown & Co., and has been laid four years.
No. 10. Section of 53 pound rail laid on Belvidere & Delaware Railroad, opposite water station at Lambertville. Made by John Brown & Co. in 1868.
No. 11. Section of 56 pound rail laid on Philadelphia & Trenton Railroad on tangent south of Morrisville. Made by John Brown & Co. in 1868.

No 12. Section of 53 pound rail, laid on c
Made by John Brown & Co. in 1868.
No. 13. Section of 56 pound steel rail of c
six years. Made by John Brown & Co.
No. 14. Inside rail of the same curve at th
No. 15. Pennsylvania Steel Company.
on tangent about a mile south of Bristol.
No. 16. John Brown & Co. 56 pound steel
north of Eddington Station. Has been in u
No. 17. Section of 53 pound rail, now in
curve of about 700 feet radius. Made by Jo
in 1868. Worn on one side three years, the
the present running side, is the outside r
No. 18. Bethlehem Iron Co., 62 pounds.
south of Milford Station, Belvidere & D
years.
No. 19. Inside rail of same curve at same
No. 20. Bethlehem Iron Company, 62
above. Was worn five years on the right-h
been worn a year on the present running s
No. 21. Bethlehem Iron Company, 62 pou
tangent south of Milford station, where all



d on outside of curve at South Trenton Depot.

ail of curve at Bristol Station. Has been in use
Co.
ve at the same point.
any. A Welch, No. 2. 56 pound steel rail. Laid
stol. Has been worn four years.
d steel rail. Laid on tangent about 2,000 feet
en in use about five years.
ow in track near north end of Bergen Cut, on
e by John Brown & Co., Sheffield, England. Laid
are. then turned. and has been worn for three on
tside rail of curve.
ounds. Outside rail on curve, about two miles
re & Delaware Railroad. Has been in use six

at same point,
y, 62 pounds, outside rail on same curve as
right-hand side of the rail, then turned, and has
ning side.
. 62 pounds. Has been in use six years on long
ere all trains run at full speed.

No. 22. Bethlehem Iron Company, 62 pounds. Has been in use six years at south end of platform Milford Station. Belvidere & Delaware Railroad.

No. 23. Bethlehem Iron Company. 62 pounds. Has been in use seven years on tangent 1,500 feet north of Frenchtown Station Belvidere & Delaware Railroad.

No. 24. Bethlehem Iron Company, 62 pounds. Has been used seven years on tangent 500 feet south of Frenchtown Station, Belvidere & Delaware Railroad.

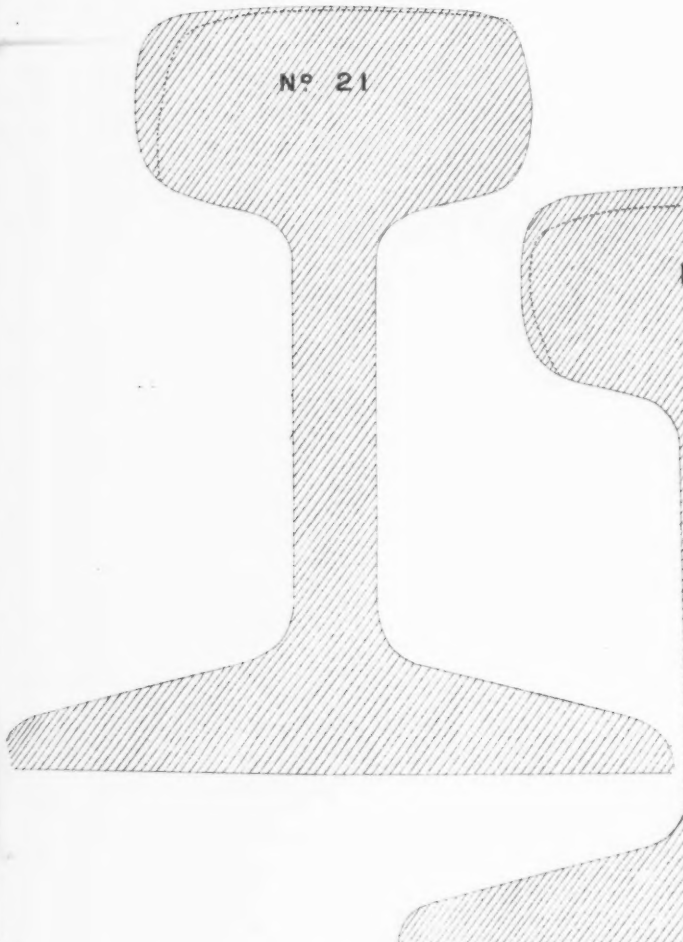
No. 25. Bethlehem Iron Company, 62 pounds. Has been worn seven years on tangent 1,500 feet south of Frenchtown Station, Belvidere & Delaware Railroad.

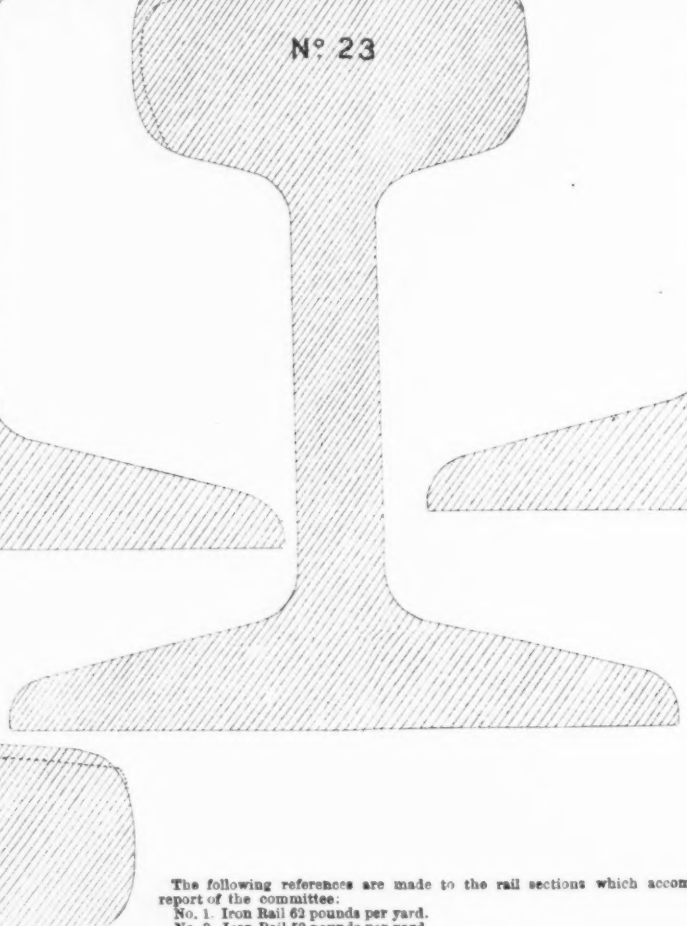
No. 26. Bethlehem Iron Company, 62 pounds. Laid in the fall of 1869 on outside of curve about half a mile south of Bristol Station.

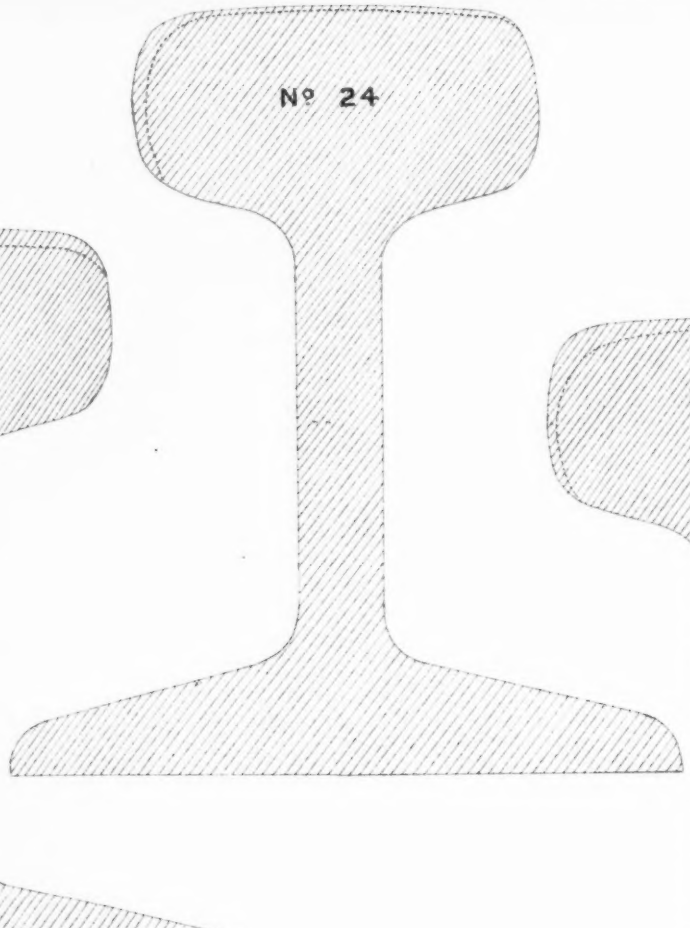
No. 27. Inside rail of same curve.

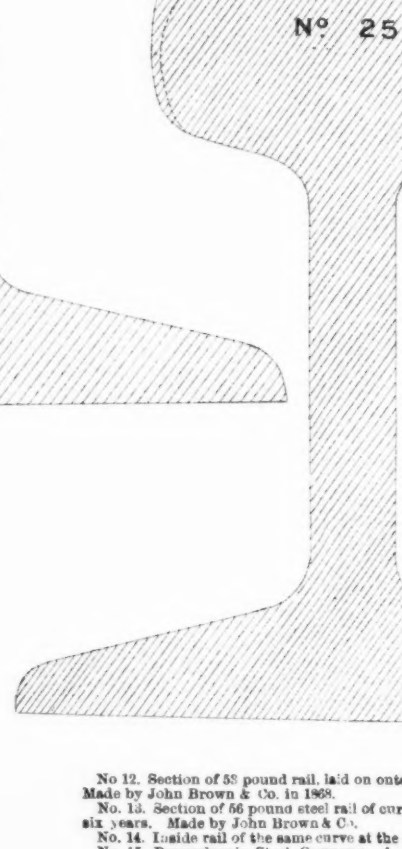
No. 28. Bethlehem Iron Company, 62 pounds. Laid in the spring of 1869 on tangent 1½ miles south of Bristol.

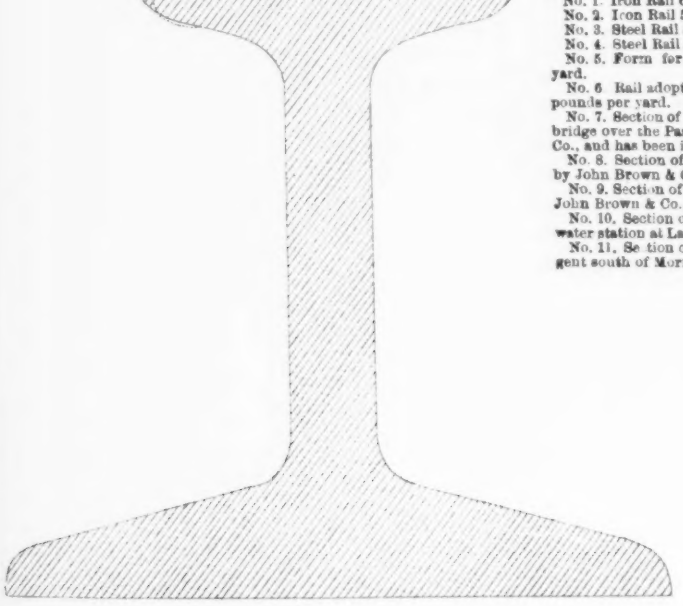
No. 29. Rail used on Lehigh Valley Railroad at Wetherly. ABCD shows the shape of head after 10 years wear. AEFD shows original shape of head as restored by calculation. Steel rail originally weighed 58 pounds per yard, reduced by wear to 46 74-100 pounds per yard. On further investigation the officers in charge of "maintenance of way," Lehigh Valley Railroad, say that the section as restored is about 1-10 of an inch too high, and that the original weight was probably only 56 pounds per yard, which would give the section shown by the dotted line. Area of the section bounded by the upper dotted line 5 6-10 square inches. Area of the section of rail as worn 4 67-100 square inches.





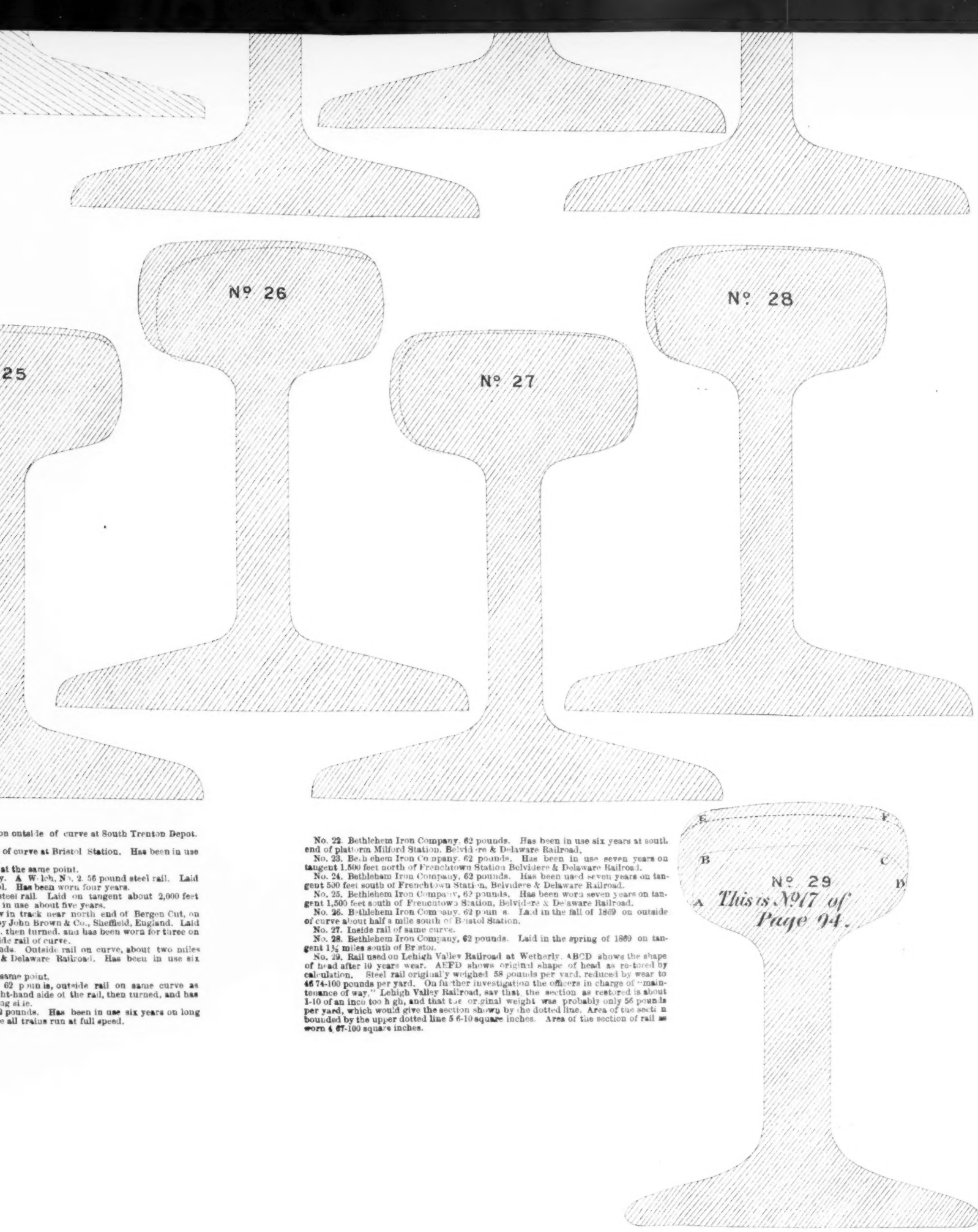






The following references are made to the rail sections which accompany the report of the committee:

No. 1. Iron Rail 62 pounds per yard.
No. 2. Iron Rail 50 pounds per yard.
No. 3. Steel Rail 53 pounds per yard.
No. 4. Steel Rail 56 pounds per yard.
No. 5. Form for Steel Rail proposed by Mr. O. Chanute, — pounds per yard.
No. 6. Rail adopted by Mr. Sayre from the Easton & Amboy Railroad, weight 65 pounds per yard. Same pattern for iron and steel.
No. 7. Section of 56 pound rail now in use on outside of curve near frog south of bridge over the Passaic River, Center street route, Newark. Made by John Brown & Co., and has been in use four years.
No. 8. Section of 53 pound rail in use on straight line in Jersey City yard. Made by John Brown & Co., and has lain four years.
No. 9. Section of 56 pound rail in use on main line at Newark Meadows. Made by John Brown & Co., and has been laid four years.
No. 10. Section of 53 pound rail laid on Belvidere & Delaware Railroad, opposite water station at Lambertville. Made by John Brown & Co. in 1868.
No. 11. Section of 56 pound rail laid on Philadelphia & Trenton Railroad on tangent south of Morrisville. Made by John Brown & Co. in 1868.
No 12. Section of 53 pound rail, laid on ont
Made by John Brown & Co. in 1868.
No. 13. Section of 56 pound steel rail of cur
six years. Made by John Brown & Co.
No. 14. Inside rail of the same curve at the
No. 15. Pennsylvania Steel Company. A
on tangent about a mile south of Bristol. Ha
No. 16. John Brown & Co. 56 pound steel r
north of Eddington Station. Has been in use
No. 17. Section of 53 pound rail, now in tr
curve of about 700 feet radius. Made by John
in 1868. Worn on one side three years, then
the present running side, is the outside rail
No. 18. Bethlehem Iron Co., 62 pounds.
south of Milford Station. Belvidere & Dela
years.
No. 19. Inside rail of same curve at same p
No. 20. Bethlehem Iron Company, 62 p
above. Was worn five years on the right-han
been worn a year on the present running si
No. 21. Bethlehem Iron Company. 62 poun
tangent south of Milford station, where all tr



on outside of curve at South Trenton Depot.

of curve at Bristol Station. Has been in use

at the same point.
y. A. Welch, No. 2. 56 pound steel rail. Laid
l. Has been worn four years.
steel rail. Laid on tangent about 2,000 feet
in use about five years.
w in track near north end of Bergen Cut, on
by John Brown & Co., Sheffield, England. Laid
. then turned, and has been worn for three on
de rail of curve.
nds. Outside rail on curve, about two miles
& Delaware Railroad. Has been in use six

same point.
62 pounds, outside rail on same curve as
ht-hand side of the rail, then turned, and has
ng side.
pounds. Has been in use six years on long
e all trains run at full speed.

No. 22. Bethlehem Iron Company, 62 pounds. Has been in use six years at south end of platform Milford Station, Belvidere & Delaware Railroad.

No. 23. Bethlehem Iron Company, 62 pounds. Has been in use seven years on tangent 1,500 feet north of Frenchtown Station Belvidere & Delaware Railroad.

No. 24. Bethlehem Iron Company, 62 pounds. Has been used seven years on tangent 500 feet south of Frenchtown Station, Belvidere & Delaware Railroad.

No. 25. Bethlehem Iron Company, 62 pounds. Has been worn seven years on tangent 1,500 feet south of Frenchtown Station, Belvidere & Delaware Railroad.

No. 26. Bethlehem Iron Company, 62 pounds. Laid in the fall of 1869 on outside of curve about half a mile south of Bristol Station.

No. 27. Inside rail of same curve.

No. 28. Bethlehem Iron Company, 62 pounds. Laid in the spring of 1869 on tangent 1½ miles south of Bristol.

No. 29. Rail used on Lehigh Valley Railroad at Wetherly. ABCD shows the shape of head after 10 years wear. AEFD shows original shape of head as restored by calculation. Steel rail originally weighed 58 pounds per yard, reduced by wear to 46 74-100 pounds per yard. On further investigation the officers in charge of "maintenance of way," Lehigh Valley Railroad, saw that the section as restored is about 1-10 of an inch too high, and that the original weight was probably only 56 pounds per yard, which would give the section shown by the dotted line. Area of the section as bounded by the upper dotted line 5 6-10 square inches. Area of the section of rail as worn 4 67-100 square inches.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

LXXXIX.

NOTES ON

THE WEIGHT OF RAILS AND THE BREAKING OF IRON RAILS.

A Paper by O. CHANUTE, C. E., Member of the Society.

PRESENTED JULY 15, 1874.*

THE WEIGHT OF RAILS.—The writer has long entertained the opinion that the assumption of any fixed relation between the weight of a rail and the maximum load borne by one pair of wheels passing over it, was misleading and fallacious. A simple consideration of the pressure on the surfaces in contact between the driving wheels of the locomotives and the rails will make this apparent. If the wheel and the rail were both inelastic, the contact would be a line, and the pressure would be infinite. Being elastic, however, they both yield a little, and the contact becomes a surface.

Ten years ago, the writer took some pains to measure these surfaces, and then found them to average about $\frac{3}{4}$ of an inch across the face of the rail, and about $\frac{3}{8}$ of an inch along its length; giving an area of $\frac{9}{32}$ of a square inch, on which a weight of 9,000 pounds will exert a pressure of 32,000 pounds per square inch—10,000 pounds, a pressure of 35,555 pounds per square inch, and 11,000 pounds, a pressure of 39,111 pounds

* Appended to the Report on the Form, Weight, Manufacture and Life of Rails, LXXXVIII, Transactions.

per square inch. These experiments have recently been repeated upon the Erie Railway. Those which seemed most satisfactory were made by jacking up simultaneously both wheels of a pair of drivers, and introducing between them and the rails a sheet of thin tissue paper, underlaid by a black sheet of thin manifold copying paper. By lowering the wheels upon the paper, and allowing their full weight to rest thereon, a very good impression of the surfaces in contact was obtained.

In the following Table the pressures are *static*, and as ordinary wrought-iron begins to crush at about 38,000 to 40,000 pounds to the square inch, the weights noted, when put into motion are pretty sure to smash down any material so soft as ordinary bar iron. While the maximum weight on the wheels, therefore, should undoubtedly govern the hardness of the *material* in the head of the rail, it has no relation to the *weight* of the rail, except so far as this results from its necessary form and proportion as a girder to carry and distribute the weight to the supports, and this again is modified by the distance these may be spaced apart.

No. of engine.	Diameter of driver.	Weight on driver.	Position of wheel.	Condition of wheel.	General size of impression.	Area in contact.	Pressure per square inch.
	ft. in.	Pounds.			Inches.	Inches.	Pounds.
388	5 0	11,175	Right.	Worn.	$\frac{7}{16} \times \frac{1}{4}$	0.13	85.961
"	5 0	11,175	Left.	Much worn.	$2 \times \frac{3}{16}$	0.42	26.607
482	4 7½	11,358	Right.	Newly turned.	$\frac{3}{8} \times \frac{1}{4}$	0.32	35.494
"	4 7½	11,358	Left.	Newly turned.	$\frac{7}{16} \times \frac{1}{4}$	0.14	81.128
492*	5 0	11,912	Right.	Partly worn.	$\frac{3}{8} \times \frac{1}{16}$	0.36	33.089
"	5 0	11,912	Left.	Partly worn.	$\frac{3}{8} \times \frac{1}{8}$	0.35	34.034
384	5 0	11,185	Right.	Partly worn.	$\frac{1}{2} \times \frac{1}{4}$	0.32	34.953

If these premises are correct, it follows that the adding of weight to an iron rail, already sufficiently stiff to carry the load, would only ensure its more rapid destruction, by making it too rigid to yield slightly under the wheel, and perhaps by preventing such slight deflections between the supports, as is found admissible in practice, and which by curving the rail to some assimilation with the wheel, probably increases slightly the

* The experiment on engine 492 was made by substituting thin bank-note paper for the tissue paper, and that on engine 384 by wetting the rail around the area in contact with the wheel with a solution of sulphate of copper. The copper being deposited on the rail and wheel, left a bright spot of iron untouched where the surfaces had been in contact. Neither of the latter experiments were deemed as satisfactory as those with tissue paper.

areas in contact, and so diminishes the intensity of the crushing pressures. Hence we find that in many cases, some light shallow rails, with hard material in the head, thoroughly welded, have greatly outworn much heavier iron rails, under the same traffic. Hence, also, railroads with a heavy traffic, requiring powerful locomotives, with 11,000 to 12,000 pounds per wheel, in order to haul maximum trains and keep down the expenses of their train service, really cannot afford to lay any but steel rails in their main tracks. Railroads with a moderate traffic, which cannot as yet afford to buy steel rails, and have from 10,000 to 11,000 pounds on their engine wheels, should stipulate for the hardest iron it is possible to place in the heads of their rails, with a due regard to perfect welding; while those with a light business would actually find it more profitable to sell any locomotives with more than 10,000 pounds per wheel, than to have them crush and destroy the iron rails now laid in their tracks, before the growth of the country furnishes a remunerative business.

It is therefore suggested, as of more pressing importance to many of our American railroads, that they should presently proportion their machinery to the rails now laying upon them, than to endeavor to adapt the rails to the machinery. In the building of new roads, the weight and character of the rails have been governed by financial rather than truly economical considerations. Railway presidents and directors have consulted the treasurer, rather than the engineer. The latter should therefore endeavor now, so to have the engines proportioned as not to exert crushing strains on the rails.

As the first set of rails becomes worn out, and the traffic increases, the weight of the locomotives is sure to increase, and it is right that it should do so. Rates will be decreasing at the same time that the volume of business increases, and the expenses of train service will be assuming a larger proportion of the total cost of operating, so that it will be desirable to economise, and to diminish the proportion of cost of train labor, by increasing the size of the trains. This result is inevitable, and when this second period arrives, it would be a mistake to oppose an increase in the weight of locomotives to conform with moderate practice, which then becomes both necessary and economical.

When this occurs, however, the weight upon the driving wheels should govern the *material* in the rail, rather than its *weight*, and endeavors should be directed to the selection of the best materials and proportions for the second set of rails, whether of steel or of iron, instead of looking

to a mere increase of weight, which as herein before stated, may become a positive detriment. The weight of the rail will result from its form and depth, and it should be designed as a girder or beam resting upon several supports, so as best to carry and distribute the loads on the wheels. The distance apart of the ties or supports, will be governed by their cost, by the depth assumed for the rail, by convenience in tamping and repairs, as well as by the character of the soil, of the machinery and of the traffic. It may be stated in general terms, that the best practice in this country seems to point to the laying of rails 4 to 4½ inches deep, weighing from 56 to 68 pounds per yard, upon ties spaced about 2 feet from centre to centre.

The following rules are therefore proposed as a substitute for that fixing the weight of rails as a multiple of the maximum weight upon a driving wheel.

1st. Rails are to be designed as girders resting upon several supports, on the same principle, and from the same general data as those now applied to bridges.

2d. Where locomotive driving wheels are loaded to 11,000 pounds or over, the rails should be of steel.

3d. When from any cause it is not possible to provide steel rails for such weights, the material in the heads of iron rails, should be the hardest it is possible to weld thoroughly to the body of the rail.

The Breaking of Iron Rails.—Sufficient attention has probably not been given to the age of iron rails, or rather to the amount of traffic which has passed over them as affecting their liability to fracture in cold weather.

Some years ago, the writer was in charge of the maintenance of way on a prominent Western railroad. About 71 miles of this, had been laid in 1854, with an English rail of about 58 pounds weight per yard. It was pear headed, and laid with wrought-iron chairs upon joint ties. During the winter of 1864–5, these rails which had hitherto proved amply strong for the traffic and machinery, began to break under the trains, and some 300 of them snapped upon this division during the continuance of the cold weather. This breaking ceased when spring returned, and during the ensuing summer and autumn, few or no breakages occurred.

The condition of the track, already much improved, compared with what it had been in former years, was still made better during the working season. About 15 miles were relaid with re-rolled iron, thus reducing

the amount of old rails upon this portion of the road to about 56 miles; many new ties were put in, and great attention paid to the surfacing, draining and ballasting. Notwithstanding this, as soon as cold weather set in, during the winter of 1865–6, the rails began to break under the trains at a most alarming rate. Sometimes from 50 to 100 rails would be reported broken of a night, and about 3,400 rails snapped upon this 56 miles during the continuance of the cold weather. As most of these rails were 18 feet long, this amounted to about 5¼ miles of track, or over 10 per cent. of the whole.

Immediate measures were taken to guard the safety of the trains. Passenger trains were slowed to a speed of 20 miles per hour, and freight trains to 12 miles per hour, over this dangerous portion of the line. It was divided into patrols from three-fourths to one mile in length, and watchmen were put on night and day, to walk over the track after every train, to look out for broken rails, while the section gangs were held in readiness at all times to repair or replace any breakages. These mainly occurred over the shoulder ties, or those next to the joints, and about 20 to 24 inches from the end of the rail, although sometimes the rail broke into three or four pieces under the train. The precautions taken proved so effectual, that after they were adopted, only 22 trains went off the track (of which but one was a passenger train) on this portion of the road, and in every case the trains were thrown off by rails which broke under them.

The process seemed to be as follows; the rails broke under the engine, but this almost invariably passed over in safety, the car-wheels then successively loosened the broken piece of rail, until one of them threw it out of line, when the next wheel would go off the track, and the train would be wrecked, generally towards its rear end. In the vast majority of instances (say 3,490 to 22) the broken pieces of rails were not thrown out of the track by the train which had broken them, or if so, only dropped out of line after the last car had passed, and were soon thereafter found by the watchmen, when a new rail would be put in, or the pieces secured in place, so that the next train, which was flagged, could proceed in safety.

Most of the breakages occurred when the weather moderated after a very cold snap. A comparatively mild night, after two or three days of intense cold, always showed a large increased number of broken rails in the daily reports of the watchmen. An increase was also shown when the weather turned from mild to cold, the predisposing cause seeming

to be a change in the temperature, and consequently in the internal strains of the rails. As spring and warmth returned, the rails gradually ceased to break. It is almost needless to say, that during the succeeding season, the whole of that 56 miles of track was relaid with new rails.

Incredible as this experience may seem, it is believed not to be singular. It is within the personal knowledge of the writer, that an engine without a train, with a flat driving-wheel, broke no less than 163 rails, upon another Western railroad in running 30 miles, without getting off the track herself, and that some lines have recently had an experience very similar to that above described, as to the numbers of breakages.

It seems certain that after rails have been run over, either a certain number of times or of years, they become much more brittle, and likely to break off square without warning. They may have proved to be sufficiently strong during the first few years after which they were laid, and then suddenly develop increased brittleness, and break in large numbers under the same class of rolling stock, and the same conditions of working as those which they have successfully withstood during previous winters. This has been partially counteracted on some lines by annealing repaired rails. For this purpose they have been roasted to a dull red heat, over fires made of old ties, and allowed gradually to cool, but the results have not been sufficiently satisfactory to warrant a continuance of the practice.

The reader will naturally conclude that these experiences tend to confirm the theories of so called granulation or crystallization of iron under severe strains. That a very great reduction of its tensile strength takes place under repeated and severe vibrations is abundantly proved, but an inspection of the fractures of these rails did not lead to the belief that any process at all analogous to crystallization had taken place. A minute examination of the foot of the broken rails rather conveyed the impression of bright broken threads, while the heads presented a more granular appearance. If we consider that in the operation of rolling, the original grains of the wrought-iron were gradually lengthened into fibres, and that repeated tensions and vibrations would tend to break and separate some of these fibres, especially when a change of temperature had altered the internal strains in the rails, we will probably form very nearly an idea of what really took place. The fact that the greatest number broke when the weather moderated, and the general position of the fracture over the first tie, back from the joint, led to the

belief that they probably first broke by tension in the head. The latter having greater mass than either the foot or stem, would absorb heat and expand more slowly, and therefore would be in greater initial tension, especially in the night when the counteracting effect of the sun's rays would not be present. A rail being substantially a continuous beam resting upon a number of supports, would, on the passage of a load, have its top strained in compression between the points of contrary flexure between the ties and in tension over the supports. Hence the breaking over the shoulder ties upon a slight yielding of the joint.

If this theory be correct, it follows that not only must care be exercised in obtaining a hard quality of material for the head of iron rails, in order to resist crushing, that this material must weld perfectly to the the plates beneath it to avoid lamination, but also that it must be strong iron and sufficiently worked to develop tensible strength, in order that the rail shall not become brittle in using. Curiously enough, the rails which have proved most liable to fracture within the experience of the writer have been of comparatively heavy section, and generally pear-headed. The most brittle he has ever known were a lot weighing 72 pounds per yard, originally laid upon the Portage Railroad of Pennsylvania and transferred to a Western line.

The re-rolled rails made from those taken up on the 56 miles which have been alluded to, never broke. They had a very great surplus of strength, but they proved so soft that they were crushed to pieces in 3 or 4 years, while the original stock from which they were made had lasted from 10 to 11 years in the same track. In this latter case they evidently had been worked too much and tensile strength had been developed at the expense of hardness, so that the rails were no longer able to resist the crushing pressures of the driving wheels.

XC.

THE FAILURE OF THE DAM ON MILL RIVER.

A Report by JAMES B. FRANCIS, C. E., THEODORE G. ELLIS, C. E., and WILLIAM E. WORTHEN, C. E., Members of the Society.

ADOPTED JUNE 10TH, 1874.

The Committee appointed by the Society to examine and report upon the failure, on May 16th last, of the dam on Mill river, have attended to the duty assigned them and present the following report:

The dam is situated in the town of Williamsburg, Hampshire County, Mass., on a branch of Mill river, a small stream entering the Connecticut river at Northampton. It was built in 1865 by the Williamsburg Reservoir Company, a corporation chartered for the purpose, the stock in which is held by the manufacturers whose mills are situated on the stream below; the reservoir being built for the purpose of affording them a larger supply of water during the dry season.

The dam is between 500 and 600 feet long, and about 43 feet high at the highest point near the centre, diminishing to nothing at the ends, forming a reservoir, when filled, in the valley above it, of an area of 111 acres, with an average depth of about 20 feet. At the time of the failure, the water was about 4 feet below the top of the embankment,—not an unusual height at this season,—and within a few months it has been at least a foot higher. The failure took place between 7 and 8 o'clock on the morning of May 16th last, when probably three-quarters of the contents of the reservoir escaped in about 20 minutes, or at the rate of about 60,000 cubic feet per second, destroying in its course through the steep and narrow valley below, 143 lives, and property to the amount of more than $1,000,000.

The dam consists of an earthen embankment with a longitudinal wall of stone and cement through its centre, a waste-way 33 feet wide in the natural ground at one end of the embankment, and a 16-inch pipe through the embankment and wall at the lowest point for the discharge of the water as wanted for use at the mills below.

Two or three plans for a dam at this point were prepared, and rejected by the company, on the ground of their cost; one of them by the late Mr. Stewart S. Chase, Member of the Society. The plan adopted, or

rather the specifications (for it does not appear that any plan was drawn), were prepared by Mr. Lucius Fenn, Civil Engineer, now of New Britain, Conn., who, according to the evidence reported to have been given by him at the coroner's inquest, claims to have written them under the direction of the Directors,—"he acted only as the attorney of the company in drawing up the specifications."

The work was done by Wells and Bassett, contractors. A paper, embodying contract and specifications was prepared and acted under, but not signed on account of some misunderstanding. This paper was produced at the inquest, and provides for an embankment 43 feet high above the original bed of the stream, 16 feet in width at the top, with slopes on each side of one and a half to one, and for "a good rubble wall through "the centre of the dam, longitudinal therewith, the top to be 42 feet "above the original bed of the stream on the centre line of the dam, to "be 2 feet thick, uniformly level, and increase in thickness downwards "with a bater one inch per foot, making the thickness at 45 feet from "the top, 5 feet 9 inches. The whole wall to be well banded, laid in a "substantial and workmanlike manner, and except from 10 to 15 feet "from top downward, to be well grouted with the best of hydraulic lime "and sand. From the top downward, 10 or 15 feet (the exact distance "optional with the committee) may be laid in mortar with 25 per cent. "of quick lime mixed with 75 per cent. of hydraulic lime, instead of "being grouted."

"The said Wells and Bassett agree to clear and remove from the "ground under the whole embankment all perishable materials, such as "trees, stumps, roots, and also to remove all sods, muck, and everything "of that character, the whole length of the embankment, for a distance "of 30 feet each side of the centre wall under the highest part of the "embankment, and a proportionate width where the bank is of less "height. It is agreed and understood that the top of the embankment "when finished is to be 43 feet above the original bed of the stream on "the centre line of said dam, to slope uniformly at the rate of one and "one-half feet base to one foot rise on each side, to be 16 feet wide on "top, and level from one end to the other. It is also understood that all "perishable material, of whatever nature, is to be excluded, and the dam "to be built wholly of earth and masonry, the earth to be placed in thin "layers, wet and tamped or beaten with a maul for a distance of 5 feet "each side of the centre wall. The whole embankment to be built in "layers not exceeding 5 feet each in thickness, and made as solid as pos-"sible. At no time during the progress of the work, is the earth to be "less than one and a half feet below the top of the masonry until the "wall is finished."

"For foundations of masonry, the said Wells and Bassett further "agree to dig trenches 3 feet deep, or of sufficient depth to give a firm, "hard and secure bottom for the masonry to rest upon, and which will "not allow the masonry to settle."

The contract also provides that "said dam be located in such "place as the said committee shall direct, all to be done in a thorough "and workmanlike manner, and in all respects in strict conformity with "the directions, plans and specifications, and to the acceptance of the "said committee, or the superintendent employed by the said committee "to take charge of the work."

On a personal examination of the site and remains of the dam, we find that the bottom is a very compact hard-pan, overlaid by about 2 feet of coarse gravel and a few inches of soil. The hard-pan appears to be impervious to water, or as nearly so as any soil can well be, and at the breach is but slightly washed, except near the discharge pipe, where there is a gully several feet in depth, which may, however, be due in part to the former action of the brook. The gravel overlaying the hard-pan is washed and porous. A small spring is now to be seen coming out from under the remains of the embankment above the centre wall, which is evidently superficial and from the gravel. The embankment is formed of gravel similar to that above described, but containing a little loam. It was taken from the side of the hill just above the dam. It has scarcely anything binding in its character, and is not capable of being made into a puddle which would stop the percolation of water.

From the character of the bottom and the material available for the embankment, it is obvious that the chief reliance for retaining the water must be in the cement wall and the complete union of its base with the hard-pan ; the main office of the embankment being to support and protect the wall. It will be seen from the above quotation from the specifications that nothing more was required than "trenches 3 feet deep, or of "sufficient depth to give a firm, hard and secure bottom for the masonry "to rest upon, and which will not allow the masonry to settle ;" the intention evidently being only to secure a foundation sufficient to prevent settling, as if for a building or bridge abutment, totally ignoring the vital function of the cement wall, by not adopting means to prevent the passage of the water under it. It was undoubtedly expected that the embankment would prevent the water in the reservoir from reaching the base of the wall, and to make this sure, the specifications provide "the "earth to be placed in thin layers, wet and tamped or beaten with a maul "for a distance of 5 feet each side of the centre wall." Supposing this to have been done as perfectly as possible with the material, what security could it give against a pressure of 40 feet head of water ? The remainder of the embankment on the up-stream side, being built in layers of this porous material, of 5 feet in thickness, could evidently not be depended on to supply the deficiency.

The water in the reservoir, we are informed, was always very clear, as might be inferred from the character of the water-shed which, as far as observed, consists of steep rocky pasture and woodland. There would consequently be very little deposit of sediment on the embankment, which might otherwise in time have rendered it water-tight.

The specifications are also defective in providing for slopes of only one and a half to one. They should be at least two to one on the outside and two and a half to one on the inside. Another defect is in providing for a height of the embankment only 2 feet above the top of the centre wall. The height should be sufficient to protect the wall completely from frost, which in this climate requires 5 feet of earth at least. If the earth becomes frozen to the wall, it is liable to heave and fracture it. We do not describe the discharge pipe and waste-way, as the failure in the dam appears to be unconnected with them.

In the construction of the work by the contractors, it appears that there was no sufficient inspection, so peculiarly important in a work of this description, and during great part of the time none at all, except by the Directors of the Company or their building committee, during their occasional visits. The remains of the dam indicate defects of workmanship of the grossest character. The bottom of the wall does not in all cases even rest upon the hard-pan. It was in evidence at the inquest that the wall was laid up dry and grouted 5 feet high at a time. The grouting has not completely filled the cavities, and the quality of the mortar is, some of it, very bad,—as might be expected from this mode of building.

The soil and porous gravel were only partially removed from the base of the embankment for 30 feet on each side of the centre wall, as required by the specifications.

It is probably not possible to ascertain with certainty the immediate cause of the failure, but from the evidence obtained, we can come to no other conclusion than that the water found its way under the wall at a point about 100 feet from the discharge pipe, causing a slip in the embankment on the down-stream side of the centre wall, which, being then unsupported, yielded to the pressure on the upper side, and falling over, made a breach which was rapidly enlarged by the wasting away of the embankment and the fall of other parts of the wall.

It may be asked, if this was the immediate cause, why did it not happen before, when the reservoir was at a higher level? The answer, we think would be, that there has been a gradual working out of the gravel under and near the wall, and loose places or cavities formed which, when

they had attained a certain development, would suddenly lead to the failure.

The Committee understand that the principal object of their appointment to report on this matter was to study it as an engineering work, it being conceded that usually more is to be learned from one failure than many successes; but it is obvious that this cannot be called an engineering work. No engineer, or person calling himself such, can be held responsible for either its design or execution.

According to the evidence reported, it must be considered as the work of non-professional persons. This being the case, the Committee do not think it devolves upon them or the Society to criticise its design or execution. The causes of a failure attended with such disastrous consequences cannot fail to be interesting to the members of the Society, and we have endeavored to point them out.

Mr. William E. Worthen.*—The man in charge of the dam first saw a slide of earth on the low-water side; immediately the gate was opened to release the water; while doing this another slide occurred, and in about thirty minutes the whole thing was on the move. The probability is that for years there had been some percolation through the embankment at this place, and when the earth gave way, the wall was left to withstand the pressure, and gave way also.

The work was poor unquestionably, but not so bad as it might be. The contractor was paid $86\frac{1}{2}$ for the masonry, and 33 cents for the filling in. The whole sum paid him at first was from $22,000 to $24,000, of which he testified he made $6,000 profit; the County Commissioners then obliged him to spend $10,000 more; then a work cost from $32,000 to $34,000, which might have been well done for $15,000.

The result is this: the company paid towards the educating of that contractor at least $15,000, to which is now to be added at least $1,000,000 for damages caused by the failure. Men were employed who were ignorant of the work to be done, and there was nothing like an inspection, although money and life depended upon it. I do not believe, however much we are an evolved species, that we are derived from beavers; a man cannot make a dam by instinct or intuition.

* Referring to a model in clay, gave a description of the dam before and after the failure, exhibited specimens of the rock and cement of which it was made, and of the hard-pan soil adjacent.

XCI.

RESISTANCE OF BEAMS TO FLEXURE.

A Paper by JOHN G. BARNARD, U. S. Engineer Corps, Honorary Member of the Society.

READ JUNE 10th, 1874.*

I. In every bent beam, the compressed fibre and the stretched fibre, situated at units' distance from the neutral zone, develop elastic reactions always in the same ratio as the final resistances of the material to rupture by compression and extension.

II. The final resistances to instantaneous rupture developed by flexure, whether in consequence of compression or of extension, are essentially identical with those developed by *direct* compression or extension.

III. Experience and theory united, lead us to admit that the elastic reactions always vary in proportion to the load which produces flexion—even when the load increases from that which is sufficient to overcome the permanent elasticity of the material, to that capable of producing instantaneous rupture.

IV. The *formulæ of Navier* are, therefore, perfectly applicable, even up to the case of this last mentioned rupture.

Hence, as a consequence, the hypothesis that in every bent beam, the compressed and the stretched fibres situated at units' distance from the neutral zone, develop elastic reactions which are in the same ratio as the *final resistances* of the material to instantaneous rupture by compression and extension:

1st. Explains all the phenomena ascertained by experience, of the resistance to complete and instantaneous rupture of straight beams submitted to flexure.

* An abstract of a paper on Resistance of Materials, by M. Decomble, Engineer of Ponts et Chaussées (Journal des Ponts et Chaussées, February, 1872), with comments.

2d. Enables us to determine, in all circumstances, the exact value of the co-efficients of resistance to rupture by compression and by extension; and, moreover, to determine the corresponding resistance of which a beam is capable, of which the form and the co-efficients belonging to the material of which it is made, are given.

3d. Is in complete harmony with practice (which allows the loading of beams (by dead weights), up to producing one-third the resistance to instantaneous rupture of certain materials), and with the delicate experiments of Wertheim and other experimenters, which demonstrate most clearly that the smallest loads induce a permanent deformation of the material and a consequent impairing of its elasticity, that the material may be said to possess as many elastic conditions as the different loads to which it may be subjected, up to a certain (not to be accurately defined) limit, beyond which, subjected to support the weights corresponding, it ruptures in an inappreciable time; or, more explicitly, every load, however small, even if it be but the weight of the beam itself, is capable of producing rupture, provided the trial is sufficiently prolonged.

It is evident, besides, that the above is the explanation, at the same time necessary and sufficient, of the giving way, after the lapse of centuries, of masses of ancient construction, the best sheltered from the action of the weather, and which were originally built of adequate dimensions and strength.

What reason can there then be for the hypothesis so improbable, on first examination, and yet so universally adopted, that the elastic reaction under a given weight, is always the same for extension and for compression, even when the resistance to rupture, as of cast-iron for example, is three times as great for compression as for extension?

One reason assigned is, that if a beam having the cross section of an isosceles triangle is supported by its two extremities, resting first upon the base of its cross section, and then upon the opposite vertex, and equal loads applied to the middle point, in the two cases the deflections will be equal.

The author then goes into a mathematical exposition of the case in hand, assuming E_1 and E_2 to be the moduli of elasticity for compression and extension, and finds that the hypothesis of equality for these two quantities explains the phenomenon of equal deflections, the neutral axis being, under that hypothesis, the locus of the centres of gravity of the cross sections. But, for the reasons assigned, he observes that though the hypothesis of $E_1 = E_2$ harmonizes with, and explains the phenomenon observed, it is not probably *essential* to its explanation.

He then takes up the mathematical discussion again, and shows that if R_1 and R_2 are the final resistances to compression and extension of the material, and, if in addition, the compressed and extended fibres at equal distances from the neutral zone, develop elastic efforts which are in the same proportion as R_1 and R_2, the phenomenon of equal deflections (or equal *sagittæ* for the different positions of the beam of triangular section) is equally well accounted for, as by the hypothesis $E_1 = E_2$, which latter, the author asserts to be radically discordant "with all the phenomenon which practical construction is most imperatively called upon to take account of."

And he goes on to say: "In other words, $E_1 = E_2$ corresponds, simply to a *particular case* of the general problem; a case rarely, perhaps never, encountered in practice. Hence we set aside this supposition of $E_1 = E_2$, which in all that concerns the engineer, has no practical realization, and the formulæ of Navier, grounded on the modified hypothesis, become completely applicable, even up to the *instant of instantaneous rupture*. Moreover, from Navier's formulæ, properly applied, the values of R_1 and R_2 can be determined for any material by experiment with breaking beams of the material."

The author proceeds to describe the actual application to the determination of R_1 and R_2, (and the proper dimensions for flag-stones, used in covering an aqueduct), and concludes with the remark:

"It is to be observed that the introduction, into the known formulæ, of *two* co-efficients—to resistance to rupture by compression and extension, R_1 and R_2—instead of the single co-efficient, styled 'of resistance to flexure,' changes in no degree the process of the calculations ordinarily practised by engineers, *while changing the results very considerably*, and in a manner eminently useful."

I remark, in this connection (although it may be implied from what has been said), that the manner of calculation just mentioned, involves a position of the neutral zone not containing the centre of gravity; and, inasmuch as there is no source from which the forces acting across a normal section of a beam can be derived, except from the compression and extension of fibres, I am inclined to maintain that resistance to rupture materially greater than that due to the co-efficient of elasticity, as calculated by ordinary formulæ (all essentially the same as Navier's), cannot be accounted for, except by supposing (with Hodgkinson) that the neutral zone does not coincide with the centre of gravity; and hence, without having an opportunity of investigating the grounds of Barlow's theory, I am inclined to think it untenable.

Indeed, no one can have had occasion to use the received formula derived from Navier, and which, in its ordinary form, involves the location of the neutral zone as containing the centre of gravity and a single

co-efficient, derived from the tensile strength alone, without perceiving that it gives inconsistent and unreliable results. In fact, it is founded upon two assumptions—both at variance with the results of experiment—first, the constancy of the co-efficient of elasticity (under all degrees of strain), and second, the identity of that co-efficient for compression and extension.

In Morin's Table (page 8, "Resistance des Matérianx") the co-efficient of extension for wrought-iron is, near rupture, but about $\frac{1}{2}\frac{1}{2}$d of what it is for slight tensile strains; and from the experiments* made by myself upon very large forged iron beams,† the deduced co-efficient of elasticity varies from 9,558,000 to 1,482,000 pounds, according as the beam is slightly bent or bent to the point of rupture.

Another forged beam, of identical dimensions, broke with a flexure of but one-third of that the first had undergone, and yet the calculated tensile strength was almost identical in the two cases; while the trial by direct rupture, in the testing machine at the West Point Foundry, gave results 20 and 15 per cent. smaller.

Cast-iron beams of identical dimensions were then experimented with. Of course, their flexure before rupture was but trifling compared with that of the wrought-iron beams, and the two observations of flexure, made between the application of force and rupture, gave almost identical co-efficients of elasticity. Nevertheless, specimens of the beam gave in the testing machine, a tensile strength of only two-thirds that deduced from calculation founded on the results of flexure.

It is clear enough, therefore, that the Navier formula, in the form in which we universally find it in our engineering books of reference, is very unreliable.

If, therefore, by discarding a co-efficient, founded upon an imaginary co-efficient of elasticity, and the introduction of distinct and independent factors, symbolic of resistance to rupture by compression and extension, it is shown that the Navier formula can be made reliable, an important service has been rendered to engineering science.

I have not been able to give the time necessary to such study of the papers of M. Decomble as would authorize me to do more than quote (as I have done) his general results and invite attention to the papers,

* The experiment was upon forged beams, of 12 by 15 inches cross section, which had been made for an iron target, at Fort Monroe in 1868. The beams were subjected to the action of the hydraulic press for bending armor plates, at the Brooklyn Navy Yard.

† *Vide* XIV, "Transactions," "Experiments on the Front or Shield of the experimental Casemate at Fort Monroe."

which will be found in the Annales des Ponts et Chaussées, 1857, 1867 and 1872, from which last issue I have translated the results quoted.

Mr. De Volson Wood :—This is a paper which requires much thought to make it profitable. One or two points mentioned therein have occupied my mind more or less. The writer refers to some experiments and theories of Barlow ; that the formula for rupture deduced from these, does not give results consistent with fact when a beam is strained to the point of breaking, has been long known. Barlow attempted to explain this by introducing the theory of "longitudinal flexure." After mature deliberation, I am satisfied his explanation does not meet the case, and that his theory is decidedly at fault. The cause of the discrepancy, I believe, will be found in the statement that the neutral axis does not coincide with the centre of gravity. Prof. Norton made some experiments in regard to flexure, in which he introduced the theory of "transverse shearing"—but this is not sufficient. The elements referred to by the French engineer quoted, I have considered somewhat, without knowing that he was investigating them, but I am not sure that the hypothesis named will account for the discrepancies in Barlow's theories.

The tensile strength of cast-iron is one-half to two-thirds the modulus of rupture, which agrees with the experiments of Barlow and the ratios obtained from the many experiments made.

Mr. Barnard.—In this paper I have referred to one read by me before the Society*, giving an account of experiments on an iron target for the front of a casemate at Fort Monroe.† This target resisted, without actually giving way, quite a number of shots, and among them one of the most powerful that up to that time had ever been fired. To my surprise, one of the first, and not a very heavy shot, cracked a beam in two, and some of the others were broken, all showing a marked crystalline fracture.

It was desirable to know what these beams would resist under pressure—and they were tested in a hydraulic press, the strain being applied in the centre between two supports 6 feet apart. The first beam deflected 3 inches before breaking ; the consequent extension and compression of the outer fibres was remarkable, the former approximating what Morin has given for small wire. Before the trial the beam was straight

* February 2d, 1870. XIV ; Transactions.

† The target was composed of two 12-inch wrought iron plates, 4 feet broad, one on each side of the embrasure, each plate being supported on the back by two vertical wrought iron posts or beams, 12 by 15 inches, which were bolted to the plates, and stepped above and below into granite masonry.

and of rectangular section; after fracture, the broken section was three-fourths inch narrower at the bottom than at the top, and for about 12 inches, the beam was bent to the curve of flexure. It is evident from the change of transverse section, and consequently of the location of its centre of gravity, that there was also a change in the ventral zone. The fracture was decidedly crystalline, the crystals being somewhat smaller than exhibited by the beams broken by shot.

It is almost certain that the beams, fractured by shot, did not resist anything like so much as those under the press. In the first case, there was no apparent set, although the material was the best forged charcoal iron, and without a sign of flaw or crack.

This seems to be the only experiment in which very large forgings have been broken by impact and pressure.

Mr. Robert H. Thurston.—It is singular, this difference in the effect, whether a fracture is caused quickly or slowly. The velocity of the shot, at the time of impact, must have been so great, that the period of breaking the beams was exceedingly brief. It has been shown that a rapidity of fracture changes the nature of the material, and decreases its powers of resistance.

XCII.

DRAW-SPANS AND THEIR TURN-TABLES.

A Paper by C. Shaler Smith, C. E., Member of the Society.

Read June 10th, 1874.

In presenting a paper on the above, the writer feels it his duty to state in the outset that much of what follows is intended for the "non-specialist" members of the Society, and is in answer to numerous letters of enquiry concerning these subjects.

Draw-Spans, as usually constructed, may be divided into two classes; the suspended girder or "hog-chain draw," and the continuous girder or "cantilever draw."

In the suspended girder type, there are two ordinary spans, one over each opening or bay, these spans being supported at the outer ends by suspension chains running back to a central tower resting on the turn-table. The outer ends of these suspension chains may be attached to either the upper or lower chord of the trusses. The strains in this form of draw-span are simple and unambiguous. Each girder is proportioned as for an ordinary fixed span, and—if the suspension chains are attached to the upper chord—the chains and tower will only have the easily defined strains arising from the weight of the suspended spans to resist while rotating, and this condition is not changed unfavorably when the bridge is in place and loaded, as the upper chord is shortened by the action of the load, and this relieves the tension on the chains. If, however, the suspension chains are attached to the lower chord—a member which elongates under the effects of a load—each will resist this elongation in proportion to the angle formed by chain and chord, modified by the elevation given to the ends, and the relative sectional areas of the chain and the tensile members of the lower chord, the middle section of the latter being taken as a factor in the calculation. After computing the additional stress in a chain, caused by this action of the lower chord, the former must be increased in section to meet it,

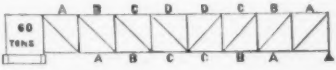
and the latter may be decreased to the same amount in its tensile members. In point of economy the suspended girder draws, if properly proportioned, are equal to the continuous girders up to about 280 feet total length, and they are more easily constructed and erected.

The continuous girder draw is now the favorite type in this country, and has been constructed in spans of a constantly increasing length, until at the Louisiana bridge one has been built 442 feet over all. The earlier bridges of this character were proportioned on the hypothesis that when opened, the truss was a beam supported at the middle, and when closed, it was resolved into two independent fixed spans. In practice, these conditions are very difficult of fulfillment, especially in the cases of large spans, as the following will illustrate. For the assumption to be correct, it is necessary that with both spans fully loaded, there should be no tension over the centre pier. A 360 feet draw weighs about 700,000 pounds, and to produce the condition required, it is necessary that the outer ends shall be raised until one-half the total weight has been elevated a distance of 6 inches (assuming the central depth of the draw to be one-tenth of the total length, or 36 feet). To do this, requires an expenditure of force to the extent of 175,000 foot-pounds, or the work of one man doing his best for 35 minutes. The cost of this work thus expended, however, is a serious objection. The formula for the spans when swung being common to all systems, another and more correct theory of calculation is to consider the bridge when fully loaded as a continuous beam resting on 3 supports. In this case, the span should be built without any allowance for camber, and the jack-screws at the ends given play enough to bring the outer pier bearings to an exact level with the bearings at the centre pier.

When a draw-span is proportioned upon this principle, the simplest method of obtaining the chord strains when the bridge is loaded from end to end, is to calculate them as for a fixed span of the same weight and load per panel, but this fixed span to be considered as *one-third longer* than the half span of the draw. Three-fourths of the strain in the middle panel of this supposititious span is the true strain over the centre pier of the real draw-span, and the chord strain in each separate panel is obtained by deducting this pier strain from each panel stress in succession. Thus, suppose the draw arm is a plain Pratt truss, 6 panels long, with single intersections, straight end posts, and ties at 45 degrees, the weight for one truss being 10 tons per panel. Now, to obtain the chord strains desired, we calculate with precisely the same data as for a fixed

span one-third longer, or 8 panels in length. The strains in this will be—panel *A* 35 tons, *B* 60, *C* 75, and *D* 80 tons—and the pier strain will be three-fourths of 80 tons, or 60 tons.

Fig. 1.



The panel strains will then be, in the true 6 panel truss:

Central panel = + 60 tons — 60 tons = — 60 tons.

Panel *A* = + 35 " — 60 " = — 25 "

" *B* = + 60 " — 60 " = ± 00 " contraflexure.

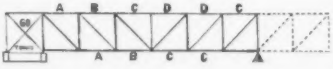
" *C* = + 75 " — 60 " = + 15 "

" *D* = + 80 " — 60 " = + 20 "

" *D* = + 75 " — 60 " = + 15 "

" *C* = + 60 " — 60 " = ± 00 " end of truss.

Fig. 2.



To check the pier strains as obtained above, multiply the span of draw arm by weight of same, and divide by 6 times the depth. Thus, in the case above, the draw arm is 6 panels long, one panel deep, and weighs 10 tons per panel, therefore, $\frac{6 \times 6 \times 10}{6} = 60$ tons.

To calculate the strains on ties and posts when the truss is fully loaded, one-third the distance from each end pier is to be considered the middle of the draw arm. To obtain the greatest counter-strains, calculate a series of strains according to the foregoing, but for the *dead load alone*, prefixing to all stresses the signs plus or minus, according as they are compression or tension. Then compute the strains for the live load alone, on one span only, as for an ordinary fixed span, and add or subtract these strains to or from those of the dead load, as the like or unlike signs may indicate.

Where one chord of a draw is entirely or partially covered by a floor, and the other is not, there is a constant variation of the elevation of the ends, owing to the uncovered member being much more rapidly affected by heat or cold than the other. So great are these changes, that the writer has seen a depression of a full inch take place in the ends of a draw 360 feet long, by 9 o'clock in the morning of a day whereon a bright, warm sun had succeeded to a cold, frosty night. At 3 o'clock in the afternoon the lower chord had become of the same temperature as the rest of the structure, and the bridge was level again. In like manner the ties and uncovered chord may lose their heat in case of a sudden cold

spell or a sleet, much sooner than the protected chord and the posts (especially where these last enclose a body of warm air). In this case, the draw arms rise instead of drooping as before. In the first instance the counter-strains in the chords, near the middle of the draw arm, are liable to become greater than calculated on, and in the second, the strains over the pier may be much increased above the theoretic strains at that point as determined by either of the foregoing systems of computation.

It is perfectly possible through this action of the temperature, combined with carelessness on the part of the attendants, for a train to come on a draw-bridge when the latter is up so high that there is practically no pressure on the end bearings. In this case, the pier strain is clearly that due to the dead load with the span swinging, plus the live load acting on a continuous beam supported on 3 points. Taking the example before cited, and considering 7 of the 10 tons per panel as live load, and 3 tons as dead load, we would have

$$\text{from dead load, } \frac{3 \times 6 \times 6}{2} = 54 \text{ tons stress.}$$

$$\text{" live " , } \frac{7 \times 6 \times 6}{6} = 42 \text{ " "}$$

$$\text{and total pier strain} = 96 \text{ " "}$$

While, if the draw had been proportioned on the first system, the theoretic strain would be; from dead load only, $\frac{3 \times 6 \times 6}{2} = 54$ tons; and if by the second system, dead and live load combined, $\frac{10 \times 6 \times 6}{6} = 60$ tons stress.

In view of these facts, the safest method of computing the stresses in a continuous girder draw-bridge is the following:

First, compute chords, ties and posts, for dead load alone; and with the draw swung.

Second, compute for live loads alone acting on a continuous beam * and add or subtract these strains to or from those first obtained, according as their signs agree or disagree.

Lastly, calculate for ties and posts as per second method, but for *live load alone*, and add or subtract these strains according as they are main or counter stresses, to or from those strains already in these members, from the dead load.

* Method given on page 131.

This will give the extreme strains when both arms are loaded from end to end, and in case a train has come on the bridge when the ends are barely touching the bed-plates.

Then compute for chords as for a continuous girder, per second method; but for *dead load alone*, and next then calculate live load only, for one draw arm, treating it as a discontinuous girder. By ranging these two series of strains together, and adding or subtracting as the signs indicate, there will be obtained the greatest possible reverse strains on the chords, at a time when from the effects of temperature the bridge has sunk on to the end bearings, and only one draw arm is loaded.

There will still remain some posts and ties, for which the extreme strains have not been found. To find these, use the ordinary formula for the rolling load, considered as advancing from abutment to centre pier, and add the strains thus obtained to those arising from the dead load already calculated for the same members, with the span swinging. Then perform the same operation, but with the load going off the bridge, and for each counter tie deduct the strain in the main tie in the same panel. This completes the work; the figures obtained by each method for each member should then be ranged in columns, and the proportions made large enough to cover the greater plus and minus strains which can come on any member under any of the varied conditions given in the foregoing. This done, the elevation given to the ends of the draw arms should be that due to the greatest possible rise and fall from the effects of temperature.

Turn-tables.—Appended to this is a Table giving the results of a series of experiments on opening and closing 11 large draw-bridges; 9 of these rest entirely on the wheel circle, the centre-pin carrying no weight whatever. In the Rock Island draw-span, the weight is mainly on the wheel circle, and the relative proportions being accurately known, the resistance is equated and computed as combined on the centre line of the wheel circle. The Arkansas bridge, however, is on a centre-pin turn-table, the wheel circle being intended simply as a steadying element. The centre-pin having settled somewhat, so that an unknown portion of the weight is borne by the wheel circle, it was necessary to consider the whole of the resistance as concentrated first on the wheel circle, to obtain the co-efficient of rolling friction in the event of that assumption being correct, and next to ascertain its value in case the weight is all on the centre-pin.

Of the draws tested, 5 had wheels of 32 inches diameter and 5

inches tread, one had wheels of 30 inches diameter and 14 inches tread, one, (the Arkansas) of 16 inches diameter and 6 inches tread, while the wheels of the remainder were of 18 inches diameter and 6 inches tread. In spite of these differences, however, there is a marked uniformity in the final co-efficient, except in the case where the strain is referred to the centre-pin.

In these tests, an accurate dynamometer was attached to the capstan bar of the driving shaft, 5 feet from the centre of the shaft (except for the Arkansas draw, where it was 7 feet), and the readings carefully taken while the draw was opened and closed a sufficient number of times to ensure a correct average of the work usually expended in this operation.*

Referring to the Table, B = multiplication of power from the end of a bar 5 feet long to the pitch line of rack circle and C, to the center line of wheels, $D = A \times B$ = power applied at the pitch line of rack circle and $E = A \times C$ = resistance at the center line of wheels, each in pounds.

The deductions drawn by the writer from these and many other experiments not here recorded (because they would render the Table too complicated), are :

First, it is unnecessary to carry any part of the weight of a draw-span to a centre-pin, for the purpose of lessening friction, as the object can be readily attained without it.

Second, for short spans, uncertain foundations, or bad masonry, the centre-pin turn-table is preferable, and for this centre-pin, a pair of plane surfaces, well oiled, and so proportioned that the weight, while moving, cannot exceed 800 pounds per square inch on these surfaces, will, in the long run, give as good practical results as any anti-friction arrangement that can be adopted.

Third, in turn-tables carrying the whole weight on the wheel circle, as good results can be obtained from 18-inch as from 30-inch wheels, provided the weight per wheel is diminished in proportion.

Fourth, the wider the tread of the wheel the greater the friction from imperfections of workmanship.

Fifth, where 2 turn-tables of precisely the same pattern, and equally well set up, are placed, one on good and the other on poor masonry, there can be as much as 50 per cent. difference in the co-efficients of friction.

* The strain of the Rock Island draw was obtained from pressures on a hydraulic jack.

ABSTRACT OF DRAW-BRIDGE TESTS, TO OBTAIN CO-EFFICIENT OF ROLLING FRICTION.

NAME OF DRAW.	Movement.	Number of readings.	Dynamometer readings in lbs. Highest.	Dynamometer readings in lbs. Lowest.	Dynamometer readings in lbs. Average = A.	Radius of pitch line of rack circle.	Radius of table, or wheel track. B.	C.	D = A × B.	E = A × C.	Feet pounds required to make 1th turn.	Total length.	Number of wheels supporting the table.	Diameter of wheels.	Weight of draw, = H.	Weight per wheel.	Friction per 1000 lbs. of superincumbent weight = E. H.	Average friction per 1000 lbs. of superincumbent weight.
						Feet.	Feet.	Feet.	Feet.					Ft. in.	Pounds.	Pounds.		
Tennessee River.....	Opening..	48	144	32	79.0	13.67	13.17	38.13	34.38	2,617	2,716	56.170	287	22	2-8	382,000	17.363	7.11
	Closing...	63	160	16	76 2	"	"	"	"	2,524	2,620	54.181	"	"	"	"	"	6.86
																		6.98
Cumberland River...	Opening..	57	128	16	64.8	73.67	13.17	33.13	34.38	2,146	2,228	46.075	284	22	2-8	381,000	17,318	5.84
	Closing...	42	112	16	63.2	"	"	"	"	2,093	2 172	44.916	"	"	"	"	"	5.70
																		5.77
Green River.....	Opening..	25	128	32	88.8	12.50	12.00	33.13	34.51	2,942	3,064	57,756	283	20	2-8	395,000	18.750	7.76
	Closing...	28	128	32	78.8	"	"	"	"	2,610	2.720	51.720	"	"	"	"	"	6.89
																		7.32
Louisville River.....	Opening..	69	115	75	80.0	14.04	13.50	33.13	34.45	2.650	2.756	58,427	264	24	2-8	522,820	21,784	5.27
	Closing...	47	130	80	82.0	"	"	"	"	2.716	2.825	59,890	"	"	"	"	"	5.40
																		5.33
Nashville.	.....	110	75	85 0	14.28	13.75	32.62	33.87	2.772	2,879	62.186	280	24	2-8½	387,000	16,125	7.43	7.43
Arkansas.....	Opening..	20	288	32	115.4	13.75	13.00	36.00	38.07	4,154	4,393	89,705	354	24	1-4	628,000	26,160	7.00
	Closing...	20	300	16	115.6	"	"	"	"	4,161	4,400	89,848	"	"	"	"	"	7.00
																		7.00*
	Opening..	20	288	32	115 4	13.75	13.00	36.00	49.50	4,154	57.123	89,683	354	24	1-4	628,000	26,160	90.96
	Closing...	20	300	16	115.6	"	"	"	"	4.161	57,222	89,838	"	"	"	"	"	91.12
																		91.04†
Rock Island.....	.....	.....	.....	.....	.....	16.50	15.00	.....	.....	10,075	10,082	261,092	368	34	2-6	1,505,000	44,264	7.36
																		7.36
Salt River.....	Opening..	92	64	8	28.1	11.64	10.97	107.14	113.68	3,010	3,194	55.032	303	34	1-6	402,000	11,823	7.94
	Closing...	115	64	8	26.5	"	"	.....	.....	2,839	3,012	51,896	"	"	"	"	"	7.49
																		7.71
White River.....	Opening..	25	32	10	19.3	11.70	10.97	68.18	72.71	1,316	1,403	24,173	281	34	1-6	403,176	11,858	3 48
	Closing...	25	32	10	19.6	"	"	"	"	1,336	1.425	24.552	"	"	"	"	"	3 53
																		3.50
Big Red River.....	Opening..	25	64	16	39.9	11.70	10.97	68.18	72.71	2,720	2,910	49.984	263	34	1-6	380,263	11,184	7.65
	Closing...	25	60	10	35.1	"	"	"	"	2,393	2,552	43.971	"	"	"	"	"	6.71
																		7.18
Little Red River.....	Opening..	25	64	10	31.7	11.70	10.97	68.18	72.71	2,161	2,305	39.715	200	34	1-6	326,808	9,612	7.05
	Closing...	25	48	28	33.2	"	"	"	"	2,263	2,414	41.593	"	"	"	"	"	7.38
																		7.21

* Co-efficient when entire weight is assumed to be carried by outer circle. † The same—carried by inner circle. (centre-pin.)

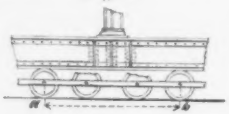
Sixth, by making the distance rim of the wheel circle vertically flexible, the excess of friction, due to imperfect masonry, can be materially diminished.

Lastly, judging from results, the best adjustment of average weight per wheel is 2,000 pounds per inch of tread at 18 inches diameter of wheel, and increasing directly as the diameter. Thus, the maximum average weight on a wheel 18 by 6 inches would be 12,000 pounds, while on a wheel 30 by 14 inches it should not exceed $14 \times 2{,}000 \times \frac{30}{18} = 46{,}666$ pounds, and on a wheel 32 by 5 inches, $5 \times 2{,}000 \times \frac{32}{18} = 17{,}777$ pounds.

The summary of the process of proportioning a turn-table in which the weight is to rest entirely on the outer wheels, is as follows. After adopting size and tread of wheel, adjust diameter of the table so that the wheels will not be overloaded. Combined with this, however, is the consideration that the span should rest on the circular girder at 4 points exactly equidistant from each other. Where possible, therefore, the central panel should be exactly the same length as the width between chord centres, and the 4 pier posts will then rest on the table in a perfect square, the diagonal of this square being the diameter of the turn-table. Thus, suppose the distance between chord centres of the trusses is 16 feet, then the centre panel should also be 16 feet, and the diameter of the wheel circle $= \sqrt{16^2 + 16^2} = 22.627$ feet. This secures absolute equality in the distribution of the weight. Should the weight become too great per wheel, or the span be too long for the diameter of the table as thus fixed, the wheel tread may be widened (a greater tread than 10 inches is not advisable) or the wheel circle may be enlarged. In this last case, the span may be carried on cross girders, resting on the quadrant points.

Assuming that the whole of the dead and live load borne by the centre pier, during the passage of a train, can be thrown upon 8 of the wheels, 2 at each corner, proportion the wheels to suit this strain, and the sectional area of the wheel-flange, as thus determined, multiplied by 1.8, will be sufficient for the tread sections of the base ring and the main circular girder. To obtain the area of the top and bottom flanges of the circular girder, it is sufficient to consider it as unsupported by the 2 wheels immediately under the main posts, and the girder as a beam extending from *a* to *b*.

Fig. 3.



In order that the load may be so distributed over all the wheels, that

those farthest from the points where the truss rests on the table shall bear while the draw is turning at least one-half of the weight borne by those immediately under the posts, it is necessary to shorten the upper flange of the table at the 4 points equidistant from the post feet, sufficient to give an upward spring to the girder under them equal to one-half the deflection due to the weight on the post, acting as a distributed load over a quarter segment of the girder considered as a straight beam, supported in the middle, and uniformly loaded. The weight of the span in forcing down this upward curvature transmits the proper proportion of pressure to the further wheels.

To obtain the power required and its most economical multiplication for hand-power, proceed as follows: for sum of resistances at centre line of wheels—*multiply, in pounds, weight of draw-span and main circle plus half the weight of wheels and spider frame, by* 7 *and divide by* 1,000, the result will be the constant of frictional resistance to be overcome at this point. Multiplying this constant by the length of a quarter segment of the wheel circle and dividing by 10,000 will give the number of minutes required for one man to open the draw.

To obtain the most economical multiplication of power, divide this constant by 40 for each man intended to be used in working the draw.

For example.—take a 360 feet draw-span weighing 700,000 pounds, diameter of table 25½ feet, the constant of resistance at wheel line is, $\frac{700{,}000 \times 7}{1{,}000} = 4{,}900$ pounds and $\frac{4{,}900 \times 20}{10{,}000} = 9.8$ minutes are required for one man to open or shut the draw. 4 men can open it in 2½ minutes, and the multiplication of power should be $\frac{4{,}900}{4 \times 40} = 30.6$; or if only 2 men can be afforded, the time of opening will become 5 minutes, and the multiplication of power $\frac{4{,}900}{2 \times 40} = 61.2$. Where steam is to be used, the gearing should be calculated for a piston speed of 200 feet per minute, with 50 pounds pressure in the cylinder.

In designing a centre-pin turn-table, the weight may be carried in to the pin, either by a single cross girder, a suspended circular or square girder frame, or by either of these last resting on struts instead of being carried by rods. In either case, however, there should be a circle of steadying or trailing wheels, in order to keep the span level while turning, and this circle should have about the same diameter as would be required for a wheel-bearing table, as it is tolerably certain that sooner

or later the wheel circle will carry a large portion of the weight. The bearing surface of the centre-pin should be so large that the weight of draw and circular girder will not be sufficient to grind out the unguent while rotating, or in other words should not exceed 800 pounds per square inch.

To obtain the frictional resistance at *circumference* of centre-pin, multiply, in pounds, weight of span and circular girder plus half weight of wheels and spider frame, by 90 and divide by 1,000.

To obtain the time required for one man to open the draw, multiply the constant thus found, by one-fourth the circumference of the centre-pin and divide by 10,000.

The most economical multiplication of power is obtained as before, by dividing the frictional resistance by 40, for each man to be employed.

To illustrate—taking the same case as before, of a draw weighing 700,000 pounds, we have $\sqrt{\frac{700{,}000}{800}} \times .7854 = 33$ inches for diameter of centre-pin; and for frictional constant, $\frac{700{,}000 \times 90}{1{,}000} = 63{,}000$ pounds. One-fourth the circumferential distance is 25.9 inches, or 2.16 feet; therefore, $\frac{63{,}000 \times 2.16}{10{,}000} = 13.6$ minutes are required for one man to open the draw, and, taking 4 men as before, $\frac{63{,}000}{40 \times 4} = 400$ per cent. is the proper multiplication of power. Multiplying to 40 by means of gearing, the diameter of the rack circle to give the necessary leverage should be 27½ feet, and that this diameter is the same as required for the 25½ feet wheel-bearing table, previously calculated, is evidence that the power is sufficient for the weight, whether carried on the centre-pin or transferred to the trailing wheels by the settlement of the centre.

Where the foundations are bad, it is advisable to use a combination of the two systems with an adjustment by which a greater or less proportion of weight may be transferred from the wheels to the pin, or *vice versa*; some of the finest turn-tables in this country are built upon this principle.

In conclusion, the writer deems it proper to state in reference to the three methods of proportioning draw-spans mentioned, that while the last is undoubtedly the most correct and the safest, yet large draw-bridges built according to the two first do their work thoroughly and well, and seem likely destined to last as long as any more modern.

MR. CLEMENS HERSCHEL :—I have been much interested in the paper just read, particularly in that part which refers to the calculation of the strains in draw-bridges. I having recently completed such a process, and although the problem is an intricate one, there is no great difficulty in it. I will endeavor at some future day to contribute a paper to the Society on this subject.

MR. CHARLES MACDONALD :—I have prepared a note on the rolling friction of draw-bridges where the weight is carried entirely on the centre, which may be of interest in connection with this paper. We have before us* what many will recognize as the remains of a Sellers' centre, used extensively for turn-tables and draw-bridges to reduce friction, the material of which it is constructed is steel throughout. The conical rollers are turned to accurately fit in the grooving of the bases, or rings, the upper one of which receives the weight to be revolved, and the lower one rests on a cast-iron seat having a semispherical bearing on the lower side, which permits sufficient rocking motion to prevent cramping in the steel-box.

This particular specimen came to grief under an extraordinary press of business, at the Point Street Draw-bridge, Providence, R. I., and is submitted as an illustration of what may not be done under similar circumstances. The Point Street Bridge has been described elsewhere and it is only necessary to refer to the accompanying sketch† to understand the method adopted in transferring the weight of the bridge to the centre. It will be observed that there are 4 radial arms, at the extremities of which, the trusses are supported, they in turn are combined upon a central bell-shaped casting, which rests upon the upper portion of the steel roller box, the lower portion of the box resting upon the rocker casting above referred to. These rollers carry an average weight of 300 tons, and as there are 22 of them, each 3½ inches long, there must have been a strain of nearly 4 tons per lineal inch of roller, supposing the loading to have been uniform upon each. Mr. Sellers,‡ the maker of these centres, informs me that he considers himself perfectly secure with a load of 3½ tons per lineal inch, providing the bearing surfaces upon the upper and lower sides of the boxes are turned to fit with perfect accuracy. He says the thickness of the boxes is not sufficient to prevent any unevenness in the bearing from being transmitted through to the grooved surfaces in which the rollers revolve.

* Referring to the parts of a draw-bridge pivot presented.

† From Engineering (London), March 21st, 1873.

‡ Of William Sellers & Co., Philadelphia.

In this case the bearing surfaces on the upper and lower castings were not turned smooth in a lathe, but being dry sand castings made with special care, I considered at the time that the bearings were quite good enough. Unmistakable failure has taken place however, and I am not prepared to dispute Mr. Sellers' opinion, that one of the causes is due to imperfect fitting; but in my judgment the controlling defect is to be found in the excessive load upon the rollers.

The Point Street Draw has been opened for the passage of river craft from 100 to 175 times per day, and at certain periods of the day it is not allowed to remain closed more than 2 minutes at a time. This box of rollers was subjected to 50,000 revolutions before obtaining its present condition, and under such a load I am not surprised that it refused to remain longer on duty.

The question as to what is a safe load upon such rollers is a very important one, and a theoretical discussion of it might prove somewhat difficult, although I doubt not some of our members are quite equal to the task. Practically, I would suggest drawing in fine lines, a full-sized circle having a diameter equal to that of the roller at the centre, and tangent to it a straight line. Then by measuring the length of tangent visibly in contact with the circle, we may assume that for all practical purposes this is a plane surface capable of sustaining a load equal to say 16 tons per square inch, which is not by any means excessive for the material in use. This would give for the present case a load of 2 tons per lineal inch of roller, about half what it has been subjected to, and quite within a safe limit, even for cases like the Point Street Draw-bridge, where the motion is almost continuous at times.

It is not necessary for me to add anything by way of commendation to the admirable contrivance Mr. Sellers has given us in these steel centres; they are too well known to require that. Failure in the present case can only be attributed to excessive loading, and possibly, as Mr. Sellers suggests—to imperfect application of the load.

Since presenting the above I have seen a theoretical investigation of the strength of friction and expansion rollers, published in the Journal of German Engineers for April, 1874, Berlin. The resultant formula from which the value of an applied load may be obtained is therein expressed as follows:

$$P = \sqrt{\frac{32}{9}\left(\frac{1}{E_1} + \frac{1}{E_2}\right) k^3 l^2 r}$$

In this, P is the load which may with safety be applied upon a

cylindrical roller whose radius is r and length l, E_1 and E_2 are moduli of elasticity of the roller and bed-plate respectively, and k is the safe compressive strain per square inch that can be sustained by the material of which the roller is composed. If we apply this formula to the case before us, we have for the constants $l = 3\frac{1}{2}$ and $r = 1\frac{1}{2}$ inches. $E_1 = E_2 = 29{,}000{,}000$ and $k = \frac{Elastic\ Limit}{2} = 35{,}000$. Substituting these values in the equation, and

$$P = 13{,}900 \text{ pounds},$$

or very nearly 2 tons per lineal inch of roller. As the actual load upon the nest of rollers, supposing the weight to have been uniformily distributed, was about double this amount, it will be seen that they were strained nearly up to the limit of elasticity of the best of steel, and the fact that they lasted so long as they did under this strain may be taken as experimental evidence of the accuracy of the theoretical results above given.

In assuming values for E and k, I have been guided by the results obtained by Capt. J. B. Eads in his experiments upon steel for the St. Louis Bridge, and for the value of r, I have taken a mean between the two radii of the conical-shaped roller, it being in diameter, $2\frac{1}{2}$ inches at the small, and $3\frac{1}{2}$ inches at the large end.

XCIII.

THE FOUNDATIONS FOR THE BROOKLYN ANCHORAGE OF THE EAST RIVER BRIDGE.

A Paper by FRANCIS COLLINGWOOD, C. E., Member of the Society.

READ JUNE 10TH, 1874.

The front face of the Brooklyn anchorage of the East River Bridge is 930 feet from the centre of the tower. The base at the front and for 85 feet 6 inches back, has a width of 109 feet 4 inches. At this point, the width is increased 10 feet for an additional length of 46 feet 6 inches, the total length being 132 feet, and the extreme width 119 feet 4 inches. The base is a timber platform of 3 feet thickness, thoroughly bolted. The lower course is laid transversely, with 1-inch spaces, the second longitudinally, with 2-inch spaces, and the third transversely, with 4-inch spaces. All spaces are thoroughly filled with cement mortar or concrete, and the platform is entirely surrounded by concrete. Below this platform are bearers placed longitudinally, with about 9-feet spaces; the bottom of these being at the level of high tide in the East River.

The surface of the ground at the front edge had an average elevation of 22 feet above tide, and at the rear, an average of 25 feet, increasing however, to 28 feet at one corner, where it came quite near to a large brick building. The west side had an elevation increasing from 28 feet to 32 feet, with buildings within 10 feet; changing suddenly down to 14 feet, however, in the rear of another building, but 6 feet away.

The earth was found, by testing, to vary quite irregularly, from loam and gravel, to sand of all kinds. Water stood in the soil at about 2 feet 4 inches above tide. At this level there was very fine sand, which, when saturated, would run like water, but was very hard and compact until disturbed, and very difficult to penetrate.

The excavation was made of such size as to give at the bottom a clear space of 18 inches between the timber platform and the sheeting all around, or 6 inches between the platform and string-pieces supporting

the lower range of sheeting. The extreme size of the excavation at the bottom was therefore 122 feet 4 inches width at the rear, 112 feet 4 inches at the front, and 135 feet long. This space had to be excavated entirely to a uniform level, before the foundation could be started; and the problem was, to so support the banks as to effectively prevent damage to surrounding property, and at the same time not have the bracing interfere with the free movements of workmen, or with lowering and placing the timber and stone in position.

The designing and carrying out of the work was intrusted to the writer, and it is hoped that it has sufficient novelty to make it worthy of record.

The design was affected somewhat by the fact, that all materials for the anchorage had to be brought from the dock at the river, 1,000 feet through crowded streets; and it was also desirable to transport for storage, the sand from the excavation to the yard at the pier. A double track tram-way of standard gauge, connecting with the street railroads was therefore laid down for one block. Two lines of rails were also provided for, over the anchorage site (connecting by switches with either track) to be put down as soon as the work would allow of it. The general level of the top of the bracing was fixed at 18 feet 6 inches above tide, to accommodate the grade of these tracks.

The form of the masonry throughout, is in plan the same as that of the foundation, the stone work being set back 18 inches from the edge of the platform all around. There are a series of offsets at the bottom, but its general form in elevation, is that of a truncated pyramid, with sides battering above ground half an inch per foot rise.

The top of the masonry is also the grade line of the bridge, and has an elevation of 89 feet at the front, and 85 feet 9 inches in the rear. The extreme size at the grade line is 104 feet 4 inches by 117 feet; the difference of 10 feet between the front and rear being preserved throughout.

The front portion, 85 feet 6 inches long, is divided into three parts. The central of these, has a width at the base of 35 feet 10 inches, and will support and contain the two central anchor chains. Between this and the two exterior walls are spaces of 13 feet 3 inches at base. Each exterior wall has a bottom width of 22 feet, and contains one of the exterior chains. The spaces will be arched over, to support the roadway above.

The position of the spaces fixed at once, approximately, that of the permanent tracks, care being taken not to interfere with the derricks

needed in prosecuting the work. The lines of bracing had also to be fixed with reference to the spaces and derricks.

As the ordinary length of spruce planks is about 13 feet, two ranges of sheeting were decided upon, at all points where the depth was not over 25 feet—and three, where the depth was greater.

This fixed at the top the extreme width of the excavation at about 126 feet, and the length, at 139 feet. As the pressure from each bank had to be carried to the bank opposite, it was necessary to divide these distances, so as to get reasonable lengths for timber, and secure sufficient rigidity to the system, without necessitating the use of inconveniently large sizes, or increasing the number of braces. It was decided that the stringers supporting the planks must be themselves supported at least once in 14 feet, and that only 8 feet spaces were desirable. Referring to the plan, it will be seen that this gave 9 divisions each in the rear and front, and 14 on each side. Since diagonal braces could be introduced as shown, this determined the use of two lines of through longitudinal bracing, and six lines of through transverse bracing.

At the intersections of the main lines, square timber piles were driven before the excavation was begun, to a depth of about 3 feet below tide. The excavation was then started at the highest point, and the first stringers were put in, as shown in section. After this was well under way, the second range of sheeting was started on the opposite side and ends, and before the pressure had become severe, the braces between the heads of the piles were put in, in each direction. In this way the work was carried down progressively; the material in the central portion being in every case the last removed.

The probable strains to which the timber would be subjected were carefully calculated, and the sizes proportioned accordingly. At three points the anchors to the derrick guys had to be partly supported by the sheeting, and the sizes of bracing were increased in order to resist the thrust occasioned thereby.

The vertical spaces between the timbers were alternately 4 feet and 2 feet. The braces, from the lowest stringers, were inclined upward, and secured to the braces next above. By this means, there was at the beginning, a clear working space of 7 feet over the greater portion of the area. This was, of course, reduced as the timber was put in. It will be seen also that the central spaces in which to lower timber, were from 12 feet to 22 feet wide and 36 feet long. This offered great facility for work, and sticks 55 feet long were lowered without difficulty.

Every precaution was taken to securely bolt the diagonals and braces, and to use cleats where there was danger of slipping; there was no failure at any point, notwithstanding some very severe rains which occured while the work was exposed. The pressure from the rear bank was sufficient to force the main braces an inch into the yellow pine piles, but no trouble arose from this cause.

When, in laying masonry, it became necessary to remove the long cross-braces, short inclined braces, reaching from the stringer to be supported, down to the masonry, were first put in and securely fastened; the long ones were then taken out with entire safety.

The sheeting was mostly 2-inch stuff; a small portion only, at an exposed point, being thicker. In driving the lowest range of sheeting, great difficulty was found in penetrating the fine compacted sand below the water line. After trying several devices, it was decided to use a water jet. For this purpose, a small rubber hose was provided, having for a nozzle a three-quarter inch iron pipe 4 feet long (with a stop-cock at its upper end). This was attached to the city water-supply, and by its use the planks were forced down very readily; 6 inches below tide was the average depth driven.

To enable us to remove the last 2 feet of the excavation, it was necessary to pump the water out of the pit; and a question arose as to the size of pump required. To solve the question approximately, the following experiment was tried. A piece of 18 inch sewer-pipe was set down into the sand at the bottom of the pit; the sand was then removed from the interior and the water bailed out. The time and depth below the top was then noted, and when the pit was nearly filled, the time was again noted, together with the increase in height. The average head under which the water entered did not exceed 6 inches, and it was thought that this was probably as great as it ever would be around the sheeting. Taking the relative perimeters of the two as a basis of comparison, gave the quantity to be pumped about 80 gallons per minute. At a time afterward, when the pump was working regularly, the amount discharged was found to be 60 gallons per minute. This method would, no doubt, be safe in similar cases where there were no springs in the bottom.

To facilitate the work of excavation, 2 piles were driven a little forward of the centre of the pit, and strongly braced to the adjacent piles, on which a temporary derrick with boom 55 feet long was erected. With this the sand boxes could be handled, and timber unloaded and

lowered into the pit with great facility. It was worked from a winding engine on the bank in the rear, and was used until the permanent derricks could be erected.

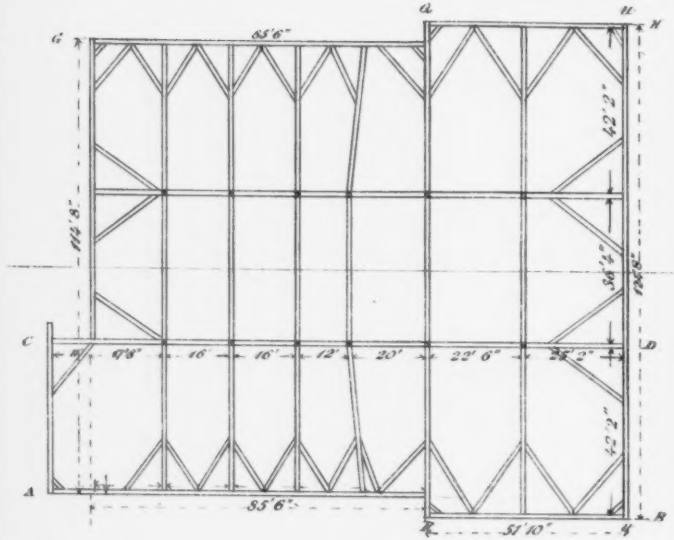
The cars were flat cars, sufficiently strong to bear a load of 10 tons. The sand boxes were very strong and proportioned to hold 2 cubic yards. They were lifted by two side chains which were fixed, and one other leading forward and fastened to the bottom, which had a tripping arrangement near the centre. The forward end was a door, with a bolt which could be tripped by the same rope which tripped the chain. The boxes were transported full of sand to the yard at the pier, where they were suspended to a traveller on an inclined wire rope, run up to an elevation of 50 feet, dumped by tripping the door and chain, lowered to the car, and again returned to the excavation. The arrangement worked well, and the cost of doing the work was moderate.

The maximum pressure upon the sand underneath, caused by the completed structure, will be about 4 tons per square foot.

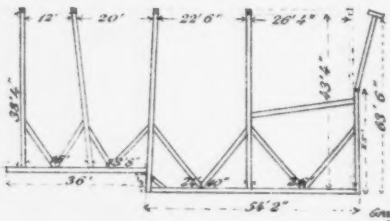
The only remaining point of interest was the method taken to lower the four anchor plates into the pit. These are massive castings, 17 feet 6 inches by 16 feet, 2 feet 6 inches deep (over all), and weighing 23 tons each. For this purpose, an excavation 20 feet wide with slope of two to one was made in the rear, and a hole cut through the sheeting. In this, timber ways were laid, and two sticks for sliding pieces were also bolted to each of the plates. The anchor plates were then lowered by tackle without trouble. This method was preferred to that of lowering direct, on account of the expensive appliances required, as well as of the risk incurred by bringing such a weight on the bracing.

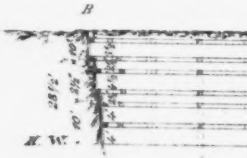
In conclusion, it is but just to say that the work would have been comparatively simple, had there been unlimited space for the operations. The extremely contracted space, and danger of undermining buildings—digging as was done 14 feet below the deepest foundation—greatly increased the risks and difficulties to be encountered. These were all overcome, however, and the masonry has been brought to the surface without damage to surrounding property, and with no mishaps from caving banks.

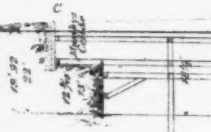
Timbers of 2nd range from Bottom of Excavation.



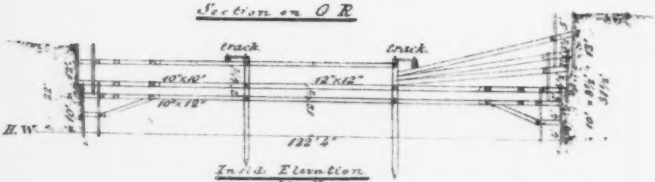
Non Timbers on West Side



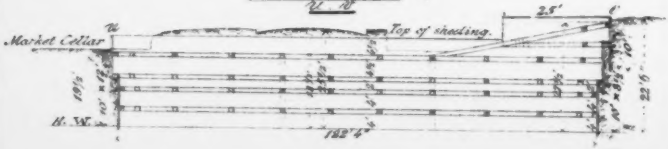




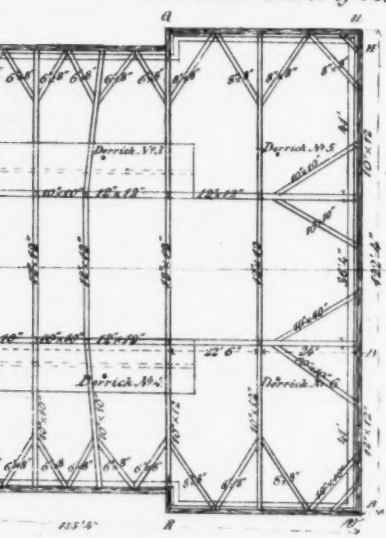
Section on O R

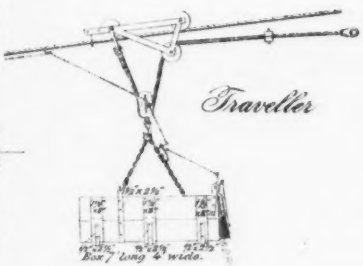


Inside Elevation of N



Lowest range of Timbers, and outlines of Timber Foundation and lower Course of Masonry.





Inside Elevation

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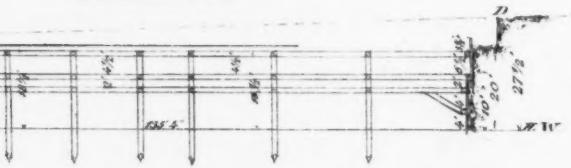
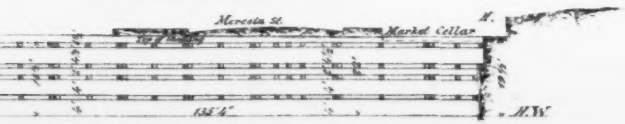


Inside Elevation



AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

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XCIV.

THE CONFLAGRATION NOW EXISTING IN THE COAL AT KIDDER SLOPE.

A Paper by MARTIN CORYELL, C. E., Member of the Society.

READ JUNE 10th,* 1874.

The mining of anthracite coal in the Wyoming valley was begun by individuals or a few men associated together to furnish the necessary capital, and a slope was sunk, through or along the coal seam, about 300 feet from the out-crop on the "pitch" or dip; then gangways (tram road ways) were turned to the right and left, following the "strike" of the seam, on a slight inclination, to the boundary of the property, which in the early days of mining was not very wide, averaging some 2,000 feet.

The "Kidder slope" was started in this way, and worked some five or six years; the demand for coal increased so rapidly, and made such promise for the investment of capital, that large companies were formed for working, smaller operations were merged into the greater, and the Wilkesbarre Coal & Iron Co. became the owners of the "Kidder colliery."

To increase production and keep pace with the demand for coal, slopes and shafts were opened on adjacent properties, and gangways ex-

* Condensed from remarks then made—at the Sixth Annual Convention.

tended on the Kidder premises far below the original workings; to facilitate these extensions, an engine with boilers was then set up some 400 feet below the surface, and connected with that on the Kidder slope, which was no longer required for pumping, hoisting coal, or for egress and ingress into the mines.

The Kidder slope and its connecting gangways had a pitch or angle of about 20° for some 600 feet, then an abrupt angle of 60° or more, and then it flattened to some 30°, upon which pitch the lower gangways were driven, connecting with the Empire shaft.

A natural and common inquiry arises—was it not dangerous, and a great risk, to generate the large quantity of steam required in the midst of so much combustible material? This, like all similar engineering questions, is answered by the fact, that it was necessary to accomplish the work; the means, therefore, had to be applied at whatever risk, with due consideration of the responsibility incurred and the advantages to be gained.

The work contemplated was accomplished and successfully carried into effect. The furnaces and boilers were securely set in brick and heavy stone masonry laid in mortar, and a brick arch flue, of some 3 feet inside diameter, and 300 feet long, was constructed to carry the hot air and gases into the old slope of the Kidder colliery. It is well known to practical men that if there is not much draft there is but little heat at the end of a flue of an anthracite furnace; and the fumes of sulphur only render a flue necessary. A passage-way was made alongside the flue, with openings, by which a watchman could pass in and examine for fire or any change of air currents. For two or more years this work went on most satisfactorily, and it was expected that by October next, that required to supercede the engine and boilers would be completed and the furnaces removed.

About December 29th last the watchman made his customary examination, and found nothing to report; two days later the men in the mines discovered something unusual, and that night the wooden stack on the surface over the air shaft, burned. This was the first intimation of the fire; the burning of the stack gave direction and intensity to the draft, which, in turn, spread the sparks and flames throughout the old workings. The miners hastened to the scene of disaster; although it does not appear they anticipated anything serious, they vigorously began to put out the fire; not that it endangered any of the present workings, but it was an unsafe element to deal with. At the surface, water from

adjacent streams was conducted to the mines by troughs and gas-pipes, and pumps were placed to use all that was available in the vicinity, whilst in the mines the mine locomotives were employed to work the steam pumps and force the water directly on the fire, which was reached by cutting passage-ways through pillars of coal and along old workings.

It will be seen by referring to the cross section presented* that the pitch of the seam is 60° or more. This is too steep to climb or work upon without ladders or scaffolding, and was sufficient for the seam to act as a chimney, giving draft and force to the fire.

The labors of the men seemed to be successful, and they exerted themselves the more; unfortunately, they spread over too much space, and left behind them smouldering and concealed fire, hid from view by the steam and smoke which filled the mine. It finally burst forth in their rear, they were surrounded by heat and gases, their supply of water and material and the means of retreat were cut off, and they escaped with much difficulty entirely exhausted by their long and severe exertions without food and sleep. By the time these overworked men could be taken from the mine, and a new force organized, the air, rarefied by the heat, passed off in furious currents, which drew their supply from all parts of the colliery, thus feeding the fire and distributing hot air and gases through the mine.

Under these circumstances a new method of proceeding had to be devised, barricades (generally termed "brattices" by miners) had to be constructed, to protect the men from the hot air and sulphurous gases, and to prevent the former from feeding the fire; when, however, the supply of fresh air was cut off from the fire, the men suffered and became helpless. It is very difficult to portray by words or by maps and profiles, to those not familiar with underground work, the many difficulties met with, so as to explain why there were many failures to one success, involving the necessity of doing the same thing over and over, under the most trying circumstances.

Those in charge reorganized their forces and prepared the material for another effort. To store up all the available supply of water, reservoirs already made were increased in capacity and others were constructed, portable boilers and steam-pumps were obtained from the cities also gas-pipe and hose in their variety, plank, boards and props and other necessary tools and materials were procured, men were placed with almost mil-

* This paper was accompanied by a set of "Plans of the Wilkesbarre Coal Co.'s Mine" showing the location and progress of the fire, which are on file at the rooms of the Society.

itary strictness and discipline, and continuously worked in shifts—in the mines of 8 hours, and outside—of 10 hours.

One of the first difficulties encountered was in the line of our profession, viz., Engineering; the early workings had been carried on when the services of the engineer were considered useless and involving unnecessary expense; there were no maps or plans to refer to, and everything done was at a venture. A roadway for the men to get at the fire had to be constructed through the old mine, every foot of the distance had to be explored and graded, without maps and profiles to show the condition of the workings where strong pillars had been reserved to make the mine secure, and where counter gangways had been opened; this seemed to be a hopeless undertaking, yet it was accomplished in a very short time, and a railroad was laid from the surface to the fire, over 3,000 feet in length, by which egress and ingress was had for men, mules and materials.

The difficulties encountered were many and great:—sometimes when lines of brattices were erected to protect the men, an undiscovered old working would let in the hot air and gases from behind. In making air-courses or roadways through pillars of coal, fire would be found burning with intensity and fierceness on the other side, thus further progress in that direction would be stopped and the labor and effort lost. In several cases it was necessary to connect sections of work which were cut off or separated by the fire, and air-courses and travelling-ways had to be made where men were exposed to the gases and intense heat. To effect this, many expedients and devices were resorted to; wooden brattices were projected into the chambers filled with hot air and glowing with heat; to maintain them in position and prevent their rapid destruction clay (if to be had) or the debris of the mine was cast against them, and the unexposed sides kept dripping with water; these brattices would soon be greatly injured if not entirely destroyed, but before this, more substantial and enduring ones would be constructed; wooden conduits—suspended on wire ropes, to keep them from the burning coal, for the conveyance of air, water, and sometimes men, were carried through the burning breasts or chambers, gases and vapor were generated in such quantities that the lamps shed but a dim, uncertain light over the tools and materials brought in and the confused mass of rubbish from former workings.

As evidence of the difficulties and danger of the work in which the men were engaged, it may be stated that a suitable position in the mine

was set apart and prepared as a hospital, a physician put in charge with all the remedies and appliances deemed essential, and a corps of men to act in time of need upon the sounding of a signal; also workmen and others were forbidden to stray off alone, and were kept in gangs of not less than three.

In the work, the men would frequently encounter what miners term "black or choke damp," and known as carbonic acid gas. When inhaled in small quantities and under certain conditions not well understood, it produces a dull pain in the head and nausea, and in large quantities, it suspends respiration and the subject drops inanimate; one of the peculiarities of its action is, that a person is overcome, without being conscious of its presence or approaching effects. It is well known that the currents of air and gases in a mine are, to use the miners term, very "baffling"—that is uncertain and beyond positive control; hence a squad of workmen pursuing their work regularly and with apparent safety, would first learn of danger from the gas by the staggering and falling of one or more of their number, a signal would be given and the corps assigned to this duty would hasten to the rescue and convey the sufferers to the hospital. In ordinary cases the head and breast of the patient were bathed in cold water and well rubbed, he was then allowed to sleep, and in half an hour would be restored and able to resume work. A bad case would be attended with vomiting, severe pains in the head and limbs and delirium; such a subject would be sent on the ambulance car to his home, and perhaps would recover after days of rest and care.

Scarcity of water was a source of difficulty, the weather became very cold, the ground froze and the streams failed to such a degree as to cause embarrassment. The heat increased in portions of the old workings where much "gob" or refuse and timber had accumulated and quantities of coal had been left in place to support the roof or on the bottom, and at the top of the seam the sulphur in the coal and slates boiled out or exuded, covering the sides and floor with a viscous substance, which, as the fire reached it, gave off volumes of sulphurous gas intensely heated, that acting as a flux, melted the sand, rocks and slates, and changed their color from a light gray to a brick red. Small explosions, supposed to be produced by quantities of water suddenly coming in contact with the ignited coal, frequently occurred, the only damage being the disturbance of the air currents.

The intensity of the fire began to manifest itself by expanding the slates and rocks over head, wide cracks or fissures appeared, and large

flakes were frequently detached; it was evident to the experienced miner that a "fall" from the roof would take place quite soon; apparently there was no immediate danger to the men, yet a little movement might cut off the communication of those down in the mine and the gases and hot air prove fatal before assistance could be rendered. At this time great progress was making in the reduction of the area on fire, all things were working to advantage, and great hopes were entertained of final success. The increasing evidence that a fall would take place caused a partial cessation of the work, particularly in that portion considered unsafe, and the men were kept out.

At night, when but few were at work, the fall took place with most disastrous results as regards the spreading of the fire; about 3 acres, averaging from 10 to 30 feet in depth, fell a distance of 10 feet at least. The fall, although not all at once, of such a mass of material (some 2,600,000 cubic feet, or 180,000 tons), was felt over quite an area of surface and with alarm to the inhabitants near by, and brought fear and anxiety into many households; no one was permitted to go near or into the mines for some time, and work was in a measure suspended. Inside the mines the damage was very great, not because of machinery or tools lost, but the brattices were all broken down and destroyed by the concussion of the air, the ground that had been reclaimed was lost, much black damp was generated and driven into the present workings, and the fire was renewed and fed by the volume of air which the destruction of brattices and air-courses permitted to enter.

In contending with the fire, it had been observed that steam concentrated and confined gave much better results than were obtained from the application of water alone, and after due consultation it was determined to inject steam into the mines, all openings being closed but one left to keep up circulation. By reference to accurate and recent surveys, "solid ground"—that is ground not broken or seriously disturbed—was selected to enclose the burning area, forming a barricade which would be proof against the fire and gases, at the same time arrangements were made whereby steam could be economically generated and admitted into the mines at points which would be available in their future working. To accomplish all that was required, it was necessary to enclose a large and very irregular area containing some 37 acres.

This work has been completed, not in a permanent manner, but so well that steam under a pressure of 60 pounds per square inch is being generated in 38 boilers (each 3 feet in diameter and 30 feet long), and

forced into the mines. The barricade against the fire is constructed in various ways; much of it is temporary, to be replaced as required. The most temporary consists of board brattices alone, and the next of board brattices and clay; a third kind is made with 2 board brattices some 4 feet apart, the enclosed space being filled in with clay or the fine dirt of the mine, and a fourth, with a wall of gob or refuse of the mine banked with clay, a mortar wall being in front. The complete and permanent brattices consist of 2 walls extending from the bottom to the top rock, made of good material laid in mortar, and the space filled in with clay; these should also sustain the roof.*

When steam was first introduced into the mine, the escaping gas and vapor indicated a temperature exceeding 220 Farh., (that being the highest graduation of the thermometer at hand, now the vapor from the enclosed area is confined to one outlet, where it escapes in not a strong current and at a temperature of about 100 . Eventually 48 boilers located as shown on the accompanying map, will be in operation to supply steam to the mines; the battery of 12—nearly completed, is to force steam down through a 9-inch hole, now being put down under contract†, and carried to a depth of some 350 feet, having about 50 feet yet to go to reach the coal seam and the fire at the foot of the inside Kidder slope. In this hole it is designed to put a 9-inch pipe, enclosing another 6-inch pipe for steam, the space between the two being for air as a non-conductor of heat.

Perhaps more might be said and the details extended, but with this paper comes a cordial invitation from the officers of the Lehigh & Wilkesbarre Coal Co.‡ (formerly the Wilkesbarre Coal & Iron Co.), for the members of the Society to personally inspect the mines, machinery and maps. Any information that this association is entitled to, will cheerfully be given, and every effort made to render a visit to these collieries instructive and agreeable. Hoping this, however inefficient of itself, will induce members to embrace the opportunity to visit the northern anthracite coal field of Pennsylvania, where nature and art presents much to see and enjoy, this paper is respectfully submitted.

Mr. J. Dutton Steele.—I have been much interested in the account of this fire. Permit me to call the attention of the Society to the very

* It may be said that a solid pillar of coal exposed on one side only to fire, and without a draft of air, will not burn.

† By the Pennsylvania Diamond Drill Co, of Pottsville, Pa.

‡ The officers are : Charles Parrish, President ; George Parrish, Superintendent; Frederick Parrish, Engineer ; John Griffith, Mine Superintendent, and David Jones, Assistant Mine Superintendent.

serious character of conflagrations in anthracite coal mines. I could enumerate 12 or 14 fires in the coal mines of Pennsylvania within a comparatively small district, that are beyond the power of the mining engineers of the present day to control or put out, some of which have been burning for 20 years, and some for but 5 or 6 years; and these are but a small part of the fires that actually take place. A large proportion, reaching below the general water level of the country are at once submerged, and after the mine rests a year or more the water is pumped out and operations resumed.

The injection of gas has been tried, but unsuccessfully; the reason is, that the mines cannot be made air-tight, which is requisite to cause the gas to reach the fire; and, of course, if a mine were air-tight, the fire would go out without gas. This plan of injecting steam is much more practicable; as a general thing, however the method pursued has been by submerging the mine as much as local circumstances allow to cut off the fire, and then to let it burn out.

If coal mining were conducted with more regard to the preservation of the product, and less to immediate profits, boilers would not be put where they are likely to ignite the mine, also less powder and other dangerous explosives would be used; as results of present management, we have many mines thus prevented from supplying one of the most valuable commodities of the country.

Mr. R. L. Cooke.—On the excursion that members at this Annual Convention are about to take, they may see one of these burning mines at Summit Hill, where anthracite coal was first discovered and supposed to be iron stones. That mine has been burning for about 15 years; the surface of the ground is sunken down and gives a fair idea of the crater of a volcano. Little puffs of smoke come up, and there are many indications of sulphur, large pieces of which are sometimes found.

XCV.

THE TEREDO NAVALIS, OR SHIP-WORM.

A Paper by G. W. R. Bayley, C. E., Member of the Society.

Read June 10, 1874.

The object of the writer in preparing this paper is, if possible, to add something to what is generally known of the habits of the Teredo, and its operations on timber structures: particularly in southern waters, on the coast of the Gulf of Mexico.

The libraries of New Orleans do not contain all of the works of naturalists and others who have written on this subject to which the writer would have liked to refer before trusting himself to prepare this paper, and circumstances have prevented the continuance of his observations respecting the habits of these "borers of the sea," and of experiments to ascertain and determine the best and most economical means of preventing their ravages upon timber foundations under water; nevertheless, what has been learned may not be entirely devoid of interest to the members of our Society, and some little may, perhaps, be new; or, at least, unknown generally.

Before proceeding with an account of what has been observed of the operations of the Teredo on the bridge-piling or foundations of the bridges on the line of the railroad, constructed in 1869 and 1870 between New Orleans and Mobile, it may not be deemed amiss to quote, somewhat at length, from what has already been published respecting the ship-worm, so far as these authorities are accessible to the writer, and are considered applicable to a better understanding of the subject.

From Louis Figuier's "Ocean World" we learn that the "Mollusca "proper have been divided by Cuvier into five sub-classes;—the first of "these is called Acephala." To this sub-class the ship-worms belong. "They have no head; the body is surrounded by the folds of the mantle; "the shell consists of two valves, hence they are called *bivalves*. * * "Although they have no head, they can feed themselves, and they re- "produce their kind. The valves are united to the animal by the inser- "tion into them of certain muscles, and the mantle-lobes stretch over "to the edges of the valves." To increase "the dimensions of its abode, "the margin of the mantle is protruded and firmly adherent all round "to the circumference of the valve with which it corresponds." The

animal "secretes calcareous matter and deposits it upon the extreme "edge of the shell, where the secretion hardens and becomes converted "into a layer of solid testaceous substance. At intervals this process is "repeated, and every newly-formed layer enlarges the diameter of the "valve."

The ship-worm moves "by means of an extensible fleshy organ "called, from some of its functions, a foot; but in fact it has less re- "semblance to a foot than to a large tongue. It is a muscular mass, "capable of being pushed out from between the mantle lobes and the "valves," and of adhering, as the writer considers, by the exhaustion of the air and water under it, firmly to the front end of the tube the Teredo is engaged in excavating or boring.

"*The Lamelli Branchiata*, so called from the leaf-like form assumed "by the branchiæ,"—or gills,—or the Acephalous Mollusca, "is devisi- "ble into two sections," of which "the Siphonida, from the animals "having respiratory siphons," includes the Teredo.

"The division Siphonida is divided into two sub-sections; those "without and those with a pallial line sinuated. The first family of "this latter section is the Pholadidæ, which includes the genera Teredo, "Xylopagha, and Pholas; animals which possess extraordinary powers "of boring, not merely through sand, but through the hardest (wood "and) rocks."

In the Proceedings of the Academy of Natural Sciences of Phila- delphia, 1862, we find it stated in "Monograph of the Family Teredidæ," "that Dr. Grey proposed, in 1851, to consider the Teredines a sub- "family of Pholadidæ, but Mr. Carpenter has separated them under the "name of Teredidæ, with great propriety: as they undoubtedly exhibit "sufficient differences from the Pholades, and from all other Mollusca, "to be entited to the position of a family."

This writer, Mr. Tryon, Jr., describes the family Teredidæ, which he divides into three sub-families; of which the second is fossil, and the third never penetrate timber, as follows: "Animal elongate, sub- "cylindrical, siphons united nearly to the end, their extremities armed "with two shelly styles—pallets, foot long and narrow, protruded "through the united mantle lobes, which are thickened in front. Gills "long; mouth with palpi. Shell, when present, globular, tripartite, in- "cluded with the animal in a more or less cylindrical testaceous tube, the "siphonal end of which is divided into two by a longitudinal partition." "Sub-family 1. Teredinæ. Valves present, free, contained in the tube, "which is irregularly cylindrical, sometimes much contorted. Per- "forating timber."

Tryon again divides the sub-family Teredinæ into the genera Teredo, Uperotis and Xylotrya. Of each genus there are many species, and, he says, it "is not unfrequently the case (that) several species are found in- "habiting the same piece of wood." There is also "great variation of

"the individuals in size, proportions, and markings," and, "with regard "to the ship-worms, * * no reliance can be placed upon the relative "proportions of their several parts for specific definition."

"So little did Linnæus and his immediate followers know," says Tryon, "of the species of Teredo, that they have included a number of "species under the name of T. Navalis, which is published with such a "general description as will suit all the species now known, or hereafter "to be added to the genus."

The writer is convinced that many species are propagated and invade timber on the coast of the Gulf of Mexico, and that they vary very much in "size, proportions and markings," some of them being from half to three-fourths inch in diameter, when fully developed, and from 12 to 20, or more inches in length,* while others are very much less in size.

As Figuier says: "the galleries bored by these imperceptible miners "riddle the whole interior of a piece of wood, destroying it entirely, "without the slightest (to an inattentive observer) external indication of "its ravages. The galleries sometimes follow the grain of the wood; "sometimes they cut it at right angles; the miners, in fact, change their "route (or retract themselves, and start again in a new place) the "moment they meet in their way either the furrows hollowed out by "their congeners, or some ancient and abandoned gallery. By a strange "kind of instinct" (not strange, because two animals cannot occupy any portion of the same tube at the same time, and they are powerless to destroy each other), "however multiplied may be their furrows or tubes "in the same piece of wood, they never mingle." He adds, that "there "is never any communication between them" in the tubes, but this is an error, for they often encroach upon each other's tubes, so far, at least, as to cut away the dividing wall between them. As, however, each animal fills the whole length of its tube at all times, for it cannot live unless its siphons and pallets (though what are the precise functions of the latter, unless to keep the outer hole open and clear and to catch the animalcules on which it feeds,† is not known) have direct connection with the sea-

* Tryon gives the dimensions of the Teredo Norvagica (whose habitat is in the waters of the Channel Islands, and Devonshire, England, coast of France, Senegal? United States? Mediterranean Sea), as follows:—"Length of valves half an inch; breadth of valves somewhat "less. Length of tube about one foot; but individuals have occurred in which the tube is "2½ feet long, and the valves three-fourths of an inch." This species, it is believed, is found near the mouth of the Mississippi River.

† When a piece of wood containing live Teredos is removed from the water, these "styles" or "pallets" are immediately protruded through the small hole through which the young Teredo entered. When replaced in water they are withdrawn. Examination, by means of a microscope, shows that these "pallets" are wholly calcareous, in shape somewhat like a "soldier's feather," imbricated, with a great number of sharp, spear-like or needle points projecting outwards, and barbed, seemingly well adapted for catching such animalcules as might float by and come within the reach of these pallets. Unless the Teredo had the means of preventing it, the shells of the barnacle or oyster would grow over this opening.

water outside; the intruder cannot advance beyond the dividing wall if the living animal be present. It is believed, however, that they do, in some cases, from necessity, cross old and abandoned galleries, when sound wood is to be found on the other side, and not elsewhere.

M. Quatrefages says that, near Port St. Sebastian, "a boat, which "served as a passage-boat between two villages, went down in conse- "quence of an accident at the commencement of spring. Four months "after, some fishermen, hoping to turn her materials to advantage, raised "the boat. But, in that short space of time, the Teredos had committed "such ravages that the planks and timbers were riddled and worm-eaten "so as to be totally useless." The writer knows of instances similar to the above which have occurred on the gulf coast of Mississippi, where submerged boats have been destroyed in as short a time. Mr. Figuier says; "at the beginning of the eighteenth century half the coast of "Holland was threatened with annihilation because the piles which sup- "ported its dikes and sea-walls were attacked by a species of Teredo." Many other cases might be cited. On the line of the railroad between New Orleans and Mobile, several hundred piles—from 14 to 20 inches in diameter, round as they grew—driven in the spring of the year, commenced to break off in the following month of November, and others, driven in June and July, commenced breaking in the following winter. Piles driven in the spring broke first at about low tide mark; those driven in summer generally broke off during the next winter, near the bottom. During the hot months the worms seemed to work, where the water was salt, nearest to the surface; later, as the temperature of the air decreased, and the surface water cooled or became less salt in consequence of rain-fall, they worked near the bottom. So completely were these heavy yellow pine piles destroyed in a few months, that the strength of one man was sufficient to break them short off, although many of them exceeded 18 and 20 inches in diameter.

As it is believed that the eggs of the Teredo are not hatched until May or June, and sometimes still later, and that the young animal does not fairly begin its operations until June or July, and sometimes in August, from three to five months seem sufficient for them to honeycomb a piece of timber of any size, during seasons favorable for their propagation and development; that is, when but little rain falls on the coast, and the sea-water is, consequently, quite salt. Pieces of very heavy yellow pine timber, 14 by 16 inches, just from the mill, were placed in the water in May and June, and taken out in the month of October following; although but 15 feet in length, they would not support their own weight, resting at both ends.

The long bridges across Bay St. Louis, 10,055 feet, and Bay Biloxi, 6,136 feet long, built in 1869 and 1870, on heavy yellow pine piles, from 15 to 20 inches and more in diameter, had to be reconstructed in the winter and spring of 1871, by driving an entire new set of piles in the place of those destroyed by the Teredo. The Bay St. Louis bridge piles—the new ones—were covered with felt and copper (or yellow metal sheathing), and the Biloxi bridge piles with felt and zinc, from the water line to the bottom; the depth at Bay St. Louis being from 10 to 12 feet generally, and at Biloxi Bay from 10 to 15 feet.

Before driving the new piles, the required depth of penetration being known by the record of the driving of the old piles, the portion intended to include the space from the water-line to the bottom was reduced to—from 10 to 12 inches in diameter and rounded, to receive the covering of felt and copper (or yellow metal), or felt and zinc, which was put on on shore. At Bay St. Louis unfortunately, or ill-advisedly, the piles were not felted and coppered for a sufficient length to allow for the increase of depth due to the scour caused by the tidal currents and the contraction of the water-way by the new piled bents. The coppering reached the old bottom, but the bottom was washed out below the protection, in many places around the piles in the new bents, thus leaving them exposed to the ravages of the Teredo below. However, as the Teredo always prefers to work lengthwise of the grain of timber, and does so invariably when he can, or except where so many animals enter as to compel their working at right angles to it in order to keep out of each other's way, those animals which attacked the *scarfed* portion of the piles below the coppering, where reduced to receive the same, naturally worked downwards with the grain and in the sapwood, thus weakening the pile but little. In those cases where the coppering did not quite reach the scarfed part of the pile there was danger of the worms entering in sufficient numbers nevertheless, to cut the pile off there or to seriously weaken it, though the animals which entered next to the copper would turn upwards with the grain of the wood.

In 1872, the writer, who was placed in charge of this road *after* the rebuilding of these two bridges across Bays St. Louis and Biloxi, employed a diver who used a submarine-armor dress, to examine each pile in both bridges. This examination showed that many of the piles in the Bay St. Louis bridge had been damaged by the Teredo below the coppering, some of them dangerously so, and that the animals were still at work in them. It became necessary to find a remedy to stop their ravages.

Pieces of wood containing live Teredos were covered under water with sand and earth, thus cutting off their connection with aerated salt-water: they died in a few days. Sand and clay were accordingly hauled in sufficient quantities and dumped around the new piles to cover the exposed parts. Although this was gradually washed away again, it remained long enough to kill the Teredos in the timber, and thus the piles were saved from destruction. The piles, much injured, were replaced with new ones, previously charred with coal-tar burnt on them, washed with dead oil, and coated with coal-tar varnish, and all loose or broken sheets of copper were renailed under water or patched with tarred canvass and copper sheets. In some few places it was found that the worms had entered above the coppering, where the piles had been driven a little too deep; these were covered higher with tarred canvass and copper. It was estimated that this coppering, already considerably thinned by decomposition, was good for, perhaps, 3 or 4 more years. As before stated, the exposed parts of the new piles in the Biloxi Bay bridge were covered with felt and zinc, instead of copper, and for a sufficient depth to allow for the increased scour. In driving the piles, many of the sheets had become loosened, as also at Bay St. Louis, by the blows of the pile-driving hammer, and but for the felting the piles would have been very imperfectly protected indeed. All the loose sheets were renailed and the broken places patched by the diver, and but few piles were found damaged seriously by the Teredo. The zincing, which had been on but one year, was found to be much corroded into small holes in many places, particularly where loosened from the felting; it was estimated that it would not last more than one or two years longer.*

The writer is confident that had the covered portion of the piles been well coated with thickened (boiled) coal-tar before the felting was put on, and the felting itself—with the same material under the metal sheets, the latter would have lasted much longer, because exposed to the salt-water only on the outside, and the felt also would have been protected from decay. As the Teredo cannot penetrate any such soft fibrous or spongy substance as felting, or thick paper, or the bark of pine timber—and this is the reason why the spongy, fibrous wood of the cabbage-tree palm, is never attacked by the Teredo—the preservation of the felt covering was certainly a matter of great importance.

* The piles in the Bay St. Louis bridge were covered with 16-ounce "yellow metal," or composition sheathing, such as is used on ships' bottoms. The Bay Biloxi bridge piles were covered with sheet zinc, numbers 9, 10, 11 and 12, weighing from 11 to 17 ounces per square foot, and some few of them with number 14 sheathing zinc, weighing 21 ounces per square foot. The lightest zinc sheets used were so corroded in one year's time, that numerous small holes had been made through them.

The action of salt water upon copper and zinc sheets is very destructive, and tarred felt is even a better protection than metal for submerged timber, because, aside from the question of durability, the former can be more closely fitted than the latter to the timber. With metal only, the sea water can penetrate under the sheets (especially where loosened or broken), and where sea water enters and ebbs and flows with the tides, so can the ship-worm enter and work. If felt, saturated with the dead oil of coal-tar and well coated with thickened coal-tar, can be secured to and maintained on the outside of submerged timber, it may be considered safe from the ravages of the Teredo. This, it is thought, can be done in several ways; by nailing with broad-headed nails, either with or without metal strips wound around spirally, or by nailing on outside of the felting, when the timber is squared or reduced to plane surfaces, thin boards with bevelled edges, which have been previously thoroughly creosoted or carbolic acidized.

It may be said that it is better to charge the whole piece of timber at once, and have done with it, but when piles from 50 to 70 feet in length must be used, as at Bays St. Louis and Biloxi, where the water was only from 10 to 15 feet in depth, with 30 or more feet of mud and clay, the cost of charging, uselessly, the portion below the bottom becomes an important consideration. In other portions of the railroad from New Orleans to Mobile, piles were required to be used from 70 to 90 feet in length, in water from 45 to 50 feet in depth; and the Teredo works to the bottom, at any known depth, in our waters.

The sap-wood portion of yellow pine timber can be easily charged with dead oil by boiling in an open tank, as the writer has determined by actual trial with round sticks from 70 to 80 feet in length, but the pitchy heart wood cannot be, by any process, in very large pieces, if at all. With round piles, unhewn, if the sap-wood is charged, and the cracks and checks are filled with thickened coal-tar, no worms can penetrate to the heart wood; it is better, therefore, to use round piles, if to be creosoted, and with the bark on if not otherwise to be protected.

The Teredo, as before remarked, works in our waters at all known depths, as at the bottom at Pascagoula, Pearl river, the Rigolets, and Chef Menteur. At those places, too, where the depths vary from 20 to 30, 40 and 50 feet, the worms do not work near the surface, where the water is fresh or brackish, but beneath it, in the salt water, and most where it is most salt, near the bottom. In some cases, they enter just above the bottom, and work their length downward, below it, as well as from just below the water-line up. The piles used to cross the shallow

lakes and bayous, in the sea marshes on the line of this railroad, from 5 to 6 feet in depth, are not injured by the Teredo, but in the open salt water Bays St. Louis and Biloxi they work nearest the surface in dry, hot weather, and nearest the bottom during cool or wet weather. During our rainy summer seasons, the Teredos do not make their appearance until late in August and September, and then are not largely propagated; but they grow several inches in length, in from four to eight weeks, when they do appear. When we have a dry summer and autumn, and the sea water is very salt, they begin their work early, and are terribly destructive. How long they live is not known, but they certainly live through our winters, and probably for one or more years, constantly increasing in size and length, if they have unperforated wood before them to bore through and grow in, and giving birth to successive families of larvæ.

It is generally conceded, now, that the process of creosoting, or "carbolic acidizing," with the dead oil of coal-tar which contains carbolic acid, is the best preservative of timber and protection against the operations of the Teredo; this has stood the test of use perfectly, since 1839, at many points on the coast of England, France and Holland. M. Crepin made extensive experiments in Belgium, and says: "in a word, "it is proven that sea water has no action on creosoted surfaces, and "that the portions of the wood well impregnated preserve their pene-"trating odor, and present no trace of alteration." The commission appointed by the Dutch Government "tried faithfully and extensively "various processes" for the preservation of wood from the attacks of the Teredo, and the conclusion was that the *creosote oil* was the only "true "preservative against the havoc to which wood is exposed on the part of "the naval worm;" and experiments in Holland, under the direction of the Amsterdam Academy of Sciences, established the same thing.

The principle question now is—not *what* is the best preservative and preventive, for that is known,—but how best and most economically to use creosote oil so called, or "dead oil," as it is known here, a product of coal-tar?

It may not be uninteresting to give some other particulars respecting the Teredo, its propagation, and how he cuts his way through the hardest wood. M. Figuier says: "The singular Acephelous Mollusc known as "the Teredo Navalis, * * has the appearance of a long worm without "articulations. Between the valves of a little shell, with which it is "provided anteriorly, may be seen a sort of smooth rim, which surrounds "a swelling projecting pad or cushion, regarded as a foot. * * The "body of the Teredo is enveloped by the shell and mantle, the latter of "which forms a sort of sheath, communicating, by two siphons with the

"exterior" sea-water. "The mantle adheres to the circumference of the "shell," and is "transparent enough, especially in the young, to permit "of the mass of the liver, the ovary, the branchiæ and the heart being "distinguished in the interior, even to counting the pulsations of the "latter. The siphons are extensile and attached, the one to the other "for about two-thirds their length. It is by these tubes that the aërated "water enters, which feeds the animal and enables it to breathe. It is "discharged by the second tube, when deprived of its oxygen and no "longer respirable, carrying with it also the useless products of diges- "tion. On the upper edge of the anterior portion of the body of the "animal is the mouth, a sort of funnel, flat and slightly bell-shaped, fur- "nished with four labial palpi, a stomach * * and a well developed "intestine."

"The heart consists of two auricles and a ventricle, which beat at "very irregular intervals, four or five in a minute. The blood is color- "less, transparent, and charged with small irregular corpuscles. The "act of breathing is accomplished in the branchiæ, or gills," which are arranged like the leaves in a book. "The nervous system is well devel- "oped and consists of nervous filaments and of ganglions which are "distributed to the mantle, the branchiæ, the foot and the siphon tubes."

"The Teredo lays a spherical greenish-yellow egg," which, "shortly "after fecundation * * is hatched. At first naked and motionless, "these larvæ are soon covered with vibratile cilia, when they begin to "move, at first * * revolving rapidly, * * afterwards swimming "about freely in water."

"When one of these larvæ has found a piece of submerged wood * * "it fabricates, step by step and as it requires them, the organs neces- "sary for the performance of its functions. It begins by creeping along "the surface of the wood * * and is observed, from time to time, to "open and shut the valves of the little embryo shell which partly envelopes "it. As soon as it has found a part of the wood fit for its purpose, it "pauses, attacks the ligneous substance, and soon produces a little depres- "sion or opening (about one twenty-fourth of an inch in diameter), which "will be the entrance to the future tunnel. Once fairly lodged in this "little opening, the young Teredo is rapidly developed; it covers itself "with a coating of mucous matter, which, condensing by degrees, assumes "a brownish tint, forming a solid covering, with two small holes for the "passage of the siphon tubes. At the end of three days this covering "has become quite solid; it is the commencement of the calcareous "tube, in which the animal is to attain its full size. When secured be- "neath this opaque screen, the little miner is no longer exposed to ob- "servation; but, if his cell is opened at the end of a few days, it is "found that it has secreted a shell, larger and more solid than the "original one, and this is the shell of the adult animal."

"The young Teredo, which feeds on the raspings of the wood (?) in- "creases rapidly; it passes first from a spheroid form to an elongated "shape," but it seems to grow very much in diameter immediately after

entering its "ligneous channel, now the dwelling-place of the animal," before extending the tube, which increases gradually in diameter, with its growth, afterwards.

Mr. Figuier says: "The process by which a creature, soft and naked "like the Teredo, should break into a solid piece of the hardest wood so "quickly, and destroy it with so much facility, was long a mystery. "Until very recently, *the shell* was looked on as the implement of perfora- "tion." He considers that, if so, "the shell should preserve certain "traces of its action upon surfaces so resistant as oak and fir," but "the "shell, on the contrary, is in such cases perfect, with no signs of fric- "tion." He thinks that "the muscular apparatus of the Teredo is not "well calculated to put the shell into rotary action, were the process a "boring one," but in this he is certainly in error.

"Some naturalists," he adds, "have suggested * * that the animal "is furnished with the means of secreting a liquid capable of dissolving "the woody fibre," but "this has been met by the statement that, in "whatever way the wood is attacked, whether the gallery is excavated "with or across the fibre, * * the groove is as exactly and neatly "cut as if it had been perforated by the sharpest tool, and that a cor- "roding dissolvent could not act with this regularity, but would attack "the harder and more tender parts unequally." M. Figuier admits that, "this objection, which M. Quatrefages opposes to the idea of a "chemical solvent," is unanswerable.

M. Quatrefages also says: "Let us not forget that the interior of the "gallery is constantly saturated with water; consequently, all the points "of the walls which are not protected by (the calcareous lining of) the "tube are subjected to constant maceration. In this state a mechanical "action, even though inconsiderable, would suffice to clear away the "bed of fibre thus softened, and, if this action is in any degree continu- "ous (as, obviously, it is and must be), it suffices to explain the excava- "tion of the galleries, however extensive their ramifications. Again, the "upper cutaneous folds, especially the cephalic hood, having the power "of expanding at will by an afflux of blood, being covered with a thick "coriaceous epidermis, and moved by four strong muscles, seems to be "very capable of performing the operation. It appears very probable "that it is this *hood* (and not the *shells*) which is charged with the re- "moval of the woody fibre, rendering it incapable of resistance by "previous maceration, which may also be assisted by some secretion "from the animal."

M. Figuier adds: "That the *fleshy parts of the mollusc*, acting upon "the surface, softened by long (?) maceration in water, is the *true boring* "*implement* employed by the Teredo, is probably the only explana- "tion the case admits of; at all events, in the presents state of our "knowledge, the explanation of this naturalist (M. Quatrefages), is the "most reasonable which can be given."

An examination of *the shells or valves* of the Teredo under a powerful microscope, by Dr. C. B. White, president of the New Orleans Board of Health and by the writer, has fully convinced them that the valves or shells are the only true implements of perforation, as first supposed; for all the striæ on the convex faces of the shells or valves, are, in fact, microscopic crystal-toothed saws, the teeth about the one thousandth of an inch high, and the same distance apart, one set of striæ or parallel saws arranged at right angles to the other, so as to cross-cut the surface to be excavated, most admirably and perfectly adapted for the purpose intended.

Before concluding our account of the boring apparatus of the Teredo, and of how it is operated, it is applicable to quote a few passages respecting its congener the Pholas, and how it operates.

"The manner in which" (says Mr. Tryon) "the animals of Pholas "excavate the holes in rocks, wood and hard clay, in which they reside, "has long proved a puzzling question to naturalists, and various theories "have been started in explanation. The hypothesis of the evolution of "an acid or solvent to eat away the surface of limestone rocks, was met "with the powerful objection that the delicate valves of the animal "itself would be equally liable to attack, and when it was found that "the Pholas, not restricting its operations to carbonate of lime, ex-"cavated with equal facility surfaces on which acid has no effect, gneiss, "for instance, 'the solvent theory' received its death blow. The use "of the valves with their sharp imbrications in effecting the work of "excavation" (he says, nevertheless), "is forbidden by their frequently "perfect state (as they should be if renewed), even when contained in "the hardest substances, (exemplified by a piece of extremely hard "gneissic rock from the coast of France, containing a magnificent "specimen of Dactylina Dactylus, with its imbricated ribs sharp and "perfect. Coll. Acad. Nat. Sci.). The anterior part of this animal, the "Pholas, has a granulated surface, caused by the presence of numerous "siliceous particles, and this is probably the instrument which the "animal employs in its work. Recent investigations have shown that "these granules *are renewed, as fast as they are worn off by attrition* with "the surrounding surface."

"De Blainville" (says Fignier) "thought that a simple movement of "the shell incessantly repeated would suffice to pierce the stone, ma-"cerated by the sea water passed through the breathing apparatus. "Mr. Robertson, of Brighton, exhibited the living Pholas in the act "of boring through masses of chalk, and thinks the process entirely "effected by the simple mechanical action of the hydraulic apparatus, "rasp and syringe."

"If you examine the living molluses," says Gosse, "you will see that "the fore part of the shell, where the foot protrudes, is set with *stony*

"*points arranged in transverse and longitudinal rows*, the former being "the result of elevated ridges, radiating from the hinge, the latter—that "of the edges of successive growths of the shell. These *points have the* "*most accurate resemblance to those set on a steel rasp* in a blacksmith "shop." The animal, Gosse adds, "turns in its burrow from side to "side when at work, adhering to the (front portion of the) interior by "the foot (or sucker), and therefore only partially rotating to and fro. "The substance is abraded in the form of a fine powder, which is gradu- "ally ejected from the mouth of the hole by contraction of the efferent "siphon."

The Teredo, probably, works in a precisely analogous manner; by adhering to the front end of his tube by means of his foot or sucker and so graduating at will the feed of his saws or rasp, which he works by a to and fro rotary motion and changing from time to time as required, the point of adhesion of the foot and the application of his transverse and longitudinally arranged system of "gang-saws" or rasps. It seems obvious, when examining the glittering crystalline teeth so systematically arranged in parallel rows on the valve shells of the Teredo, that no better arrangement could have been adopted for abrading, by a rasping to and fro rotary motion, the surface of hard wood or of anything less hard than the valve shells themselves, and not too soft or too spongy or too fibrous to be pulverized.

The Teredo, very naturally, prefers to work with the grain of timber and always does so unless prevented, because thereby the woody fibre is more easily abraded and reduced to an impalpable powder. The convex front of the shells, however, enables the animal to work at right angles to the grain of wood, or in any direction if necessity compels his doing so. He makes short and abrupt turns and tortuous windings, or sometimes even retracts himself and starts again at right angles to his gallery. He avoids knots in timber, by preference, but when crowded, the hardest and most resinous yellow pine knots are perforated with so much apparent ease as any other part of the wood. Resin is certainly no protection whatever.

A certain hardness of the wood worked in is a pre-requisite condition and without it the Teredo is powerless; therefore felt, canvas, or thick paper even, particularly if saturated with dead oil or with coal-tar—which repels the worm and preserves the material—is a protection, so far as it can be put on and maintained without breaks or openings. By nailing a sheathing of creosoted boards over the felt, canvas or paper covering, the latter would be protected and preserved, while the dead oil or coal-tar would protect the nails from oxidation.

The writer knows of an instance where the bottom of a dredge-boat, working in salt water, was protected and preserved from the operations of the Teredo by means of a sheathing of one-inch cypress boards, with a layer of felt interposed between it and the boat's bottom. Neither the sheathing nor the felt was creosoted, but, notwithstanding that the sheathing boards were destroyed or honey-combed by the Teredo, the layer of felt was not penetrated, and of course the boat's bottom was uninjured.

The Teredo is nowhere, perhaps, more destructive in its operations than on the gulf coast of Texas. Owing to the warmth of the climate there, the worm works nearly if not quite all the year round.

For the accommodation of the Morgan Line of New Orleans and Texas Steamships, a large piled wharf was constructed in 1869 at Rockport, in Aransas Bay, on the Texas coast. To protect the exposed portions of the piles of this wharf, from the high-water line to the bottom, this part of each pile was carefully charred by burning its surface. The charring of the piles—because the Teredo cannot work upon the soft charred surface—served to protect them from the Teredo temporarily, or until the barnacles had time to attach themselves and grow sufficiently to cover the surface, and thus prevent the entrance of the ship-worm. If the Teredo enters the wood in advance, there is no doubt, as before stated, that its shelly styles or pallets are used to prevent the formation of the shell of the barnacle, or of any other obstruction, over the small hole through which the Teredo supplies itself with aërated water, and with the animalcules which constitute its food. But for this provision for the protection of the Teredo, the barnacle would extend his shell over the entrance hole of the former, and suffocate it in its gallery or tube. Barnacles form on the exterior of wood containing the Teredo, it is true, but they do not cover their entrance holes so long as the animals inside are living. This crustacean, the barnacle—grows rapidly and to a large size, two inches and more in diameter—in the waters of the Texas coast, and its hard calcareous valves, ever growing and pertaining to a living animal, form a much better and more durable protection against the entrance of the Teredo, than any perishable metal covering. The difficulty seems to be, however, that protection by means of a barnacle covering is not sufficiently certain or perfect, everywhere. If the Teredo enters before the barnacle has covered the surface, (and this requires two or more months)—the latter will not succeed in covering it, as above explained. On the coast between New Orleans and Mobile,

the barnacles are generally, too few and too small to completely cover the exposed surface, except after a long time if ever, and further north they are probably, fewer and smaller still. Again, the Teredo may enter the cracks or checks in the timber, and thus avoid the charred surface.

The charred and barnacle covered piles in the Rockport wharf have been in use now about five years, and the writer has been assured, by Mr. Morgan's agent, that they have not been injured by the Teredo; as evidenced by the occasional breaking of a pile by the concussions of steamships in effecting their landings, no signs of the ship-worm then appearing.

The following simple process, in some respects the same as was adopted by the writer for the protection of the new piles used to replace the damaged felt and copper-covered piles in Bay St. Louis bridge in 1872, is recommended as an economical, and it is believed, a reliable method of protecting timber piles in salt water where exposed to the ravages of the Teredo. Place tranverse timbers or skids far enough apart to permit of the charring of the piles for the lengths required, high enough to allow for a fire of the chips made in cutting and pointing the piles underneath them, and long enough to allow for the coating of several piles and of their treatment on each side of the fire at the same time. Remove the bark (if it is on the pile) and coat the pile with coal-tar, then roll it slowly over the fire, so as to set fire to and burn the tar coating. Coat again with tar and roll back, to deepen the charred surface as well as to burn into the wood (or to impregnate it while hot with) the dead oil and carbolic acid contained in the coal-tar. After rolling off again, and as quickly as possible while the wood is yet hot, apply still another coating of tar over the charred surface, and then sift or sprinkle over all a good coating of dry beach sand, to form a firm and durable concrete crust. It would be well to roll it across the fire again, to help unite and compact the sand and tar into a concrete substance, but not to set fire to the tar again.

Coal-tar is absorbed freely when applied to a heated surface, whether wood or iron, and as Mr. Crepin says: "sea-water has no action on creosoted surfaces," while the Teredo is effectually repelled by it. Raw coal-tar applied to the bottoms of iron-hulled vessels, after the same have been well cleaned and dried, immediately after heating the metal by means of a brasier, is the best known preventive of oxidation and preservative of the metal. The coal-tar enters and fills the pores of the metal when expanded by heat and prevents oxidation. The bottoms of Mr. Morgan's iron-hulled Texas steamers are, and have been for a number

of years, periodically coated and protected in this way, and experience has fully demonstrated the value and reliableness of the method.

Cast-iron water-pipe if immersed in coal-tar while hot, have their pores filled and are coated outside, in such a manner as to prevent oxidation and preserve them when in use. Cast-iron, when to be used in or under water whether fresh or salt, would be protected from oxidation, by immersing the castings while hot or after being reheated, in coal-tar.

In like manner, timber can be partially creosoted by heating it, in the process of charring, as above described, or sufficiently so to protect it from the Teredo, particularly if in combination with the same, a concrete coating of sand and tar is formed outside.

In conclusion, it may be said that the cost of creosoting machinery as now used and the prices demanded as royalty on existing patented processes ($4 per thousand feet board measure of timber or lumber to be treated, in addition to the cost of machinery, was demanded of the writer), are in most cases, simply prohibitory. Cheaper and more expeditious methods are much needed, and it is hoped that some light has been thrown upon the subject in this paper.

A singular but a very good and substantial wharf has, it is said, been constructed of saw-mill slabs as they come from the mill, at Pensacola, Florida, by the proprietors of a saw-mill at that place. For a fixed width, the slabs are placed endwise on the bottom of the bay as nearly perpendicularly as they will stand. As new slabs are added to the front, the wharf is gradually extended from the shore line outwards into deeper water; longer slabs being used as the water deepens. The wharf is therefore nearly one solid mass of pine slabs, covered and filled in, probably, with saw-dust.

The writer is informed that the Teredo has perforated the outside slabs, at the sides of the wharf, for a foot or more into the wharf, but that all the inner portion is untouched, and, judging by what is known of the habits of the Teredo, this must be a correct statement.

The Teredo must have a connection with—in order to breathe and obtain his sustenance from—the pure aërated salt water outside of his gallery. Within such a wharf—through which there would be no current—the water would not be pure, and the animacules on which the Teredo feeds could not, without a current, enter; therefore it is probable that the Teredo would only work near to the outside.

Mr. Francis Collingwood.—I have been much interested in this paper, since it gives us reliable information on the "ship-worm" from a locality where its ravages are the greatest.

Mr. Bayley has incidentally mentioned a report made by the Amsterdam Academy of Science; a full abstract of this will be found in Van Nostrand's Magazine, Vol. IV, page 466. I desire to call attention to a report of experiments made in Norfolk harbor, by order of the United States Government; the conclusions from which will be found in the Annual of Scientific Discovery for 1857. Boxes and cubes of wood of various kinds (both protected and unprotected) were placed in the water, and subjected to frequent microscopic examinations.

The conclusions reached were as follows. Not a trace was discovered of the animal before June 20th, nor of any beginning to work later than September 29th. The animals at first appearance seem to be perfectly formed, and during the summer reach a length of 6 to 12 inches, and a diameter of $\frac{1}{8}$th inch. The cold weather usually destroys them, although some specimens always live through the winter. In Charleston harbor they develop through nearly the whole year; in New York harbor the season is somewhat shorter, while at Boston and Portsmouth the worm does but little damage. Their ravages are always less in muddy than in clear water, free from filth; and they cannot live in stagnant water. They never bore through one plank or board, except when it is in close contact with another. The bark of trees is a protection where it is entire. Other conclusions are the same as those stated by Mr. Bayley; the animal being microscopic, the entire surface must be completely covered to prevent its attacks.

The Seely process for creosoting is recommended as the most reliable of all. By this, the wood is placed in a bath of creosote oil, and exposed to a temperature of about 300°; after the water has been expelled, the hot oil is quickly replaced by cold oil; this condenses the steam in the pores of the wood, which are then thoroughly filled by the creosote.

Mr. Ashbel Welch.*—A convenient way to preserve piles in salt water from this worm, without resorting to the expensive processes recommended, is, after the pile is driven, to put a thin casing of wood or sheet metal around it, leaving, say an inch space between the two, which fill with gravel. In a short time this will harden and form a coating about the pile which no Teredo or other kind of worm will put his teeth into. The casing of course, will soon decay or corrode, and disappear.

Mr. Julius W. Adams.—Does not the preservation of piles in salt water depend more on the absence of the ship-worm rather than on

* At the Evening Session, in the discussion on "The Improvements of the Water Front of New York." XCVI, Transactions.

any precaution which may be taken? I understand that in the very heart of the timber caisson, launched at Greenpoint, brought down and sunk for the Brooklyn tower of the East River Bridge, when pieces were cut out to replace what was injured by the fire, a live Teredo an inch long, was found. This would seem to show that unless the protection or casing is put on before the pile is driven, the worm gets in, and though it cannot be supplied with salt water yet it lives.

Mr. Collingwood.—This must be a mistake: it is stated by all those who have observed the habits of the Teredo, that it dies when removed from salt water, and in the instance cited the water in the caisson must have been there a long time and the worm dead.

Mr. John D. Van Buren, Jr.—At Hoboken ferry there is a section of a built-up column, planned by myself for the wharves of this city, cased with clay pipe and concrete as a protection against the Teredo. This casing may be bored through by blows of a sledge-hammer, without being cracked. It would seem to be reliable and cheap when applied to columns or piles of large diameter, not liable to spring.

Mr. Welch.—Unless the pile is forced down by pressure as is done at Hoboken instead of by blows, it cannot until in place, be protected in the manner I described. I have had much experience with these worms, and can confirm what has been said of the effects of fresh water in this vicinity. It is the practice with the boats running between New York and Amboy, when the Teredo is found at work on one of them, to take it into the stream towards New Brunswick, where in from 12 to 24 hours, the worms are killed by the fresh water.

Mr. J. Dutton Steele.—It is stated in the paper read, that although these borers did not penetrate beneath the mud, there was practically no limit to the depth at which they worked. Is this so?

Mr. Welch.—I have known them to reach 18 inches below the mud, always entering above.

Mr. Collingwood.—They do not enter the pile below the mud, but may follow the fibre downward. It is stated by all writers on the subject, that the worm must be able to reach clear water with its syphons, which is the reason why it keeps open the passage it makes. Being a mere speck, the smallest break or crack in any protection permits its entrance, hence driving nails or spikes over the surface of the timber exposed is a failure.

XCVI.

THE IMPROVEMENT OF
THE WATER FRONT OF THE CITY OF NEW YORK.

A Paper by JOHN D. VAN BUREN, Jr., C. E., Member of the Society.

READ JUNE 10th, 1874.

CHARACTERISTICS OF THE HARBOR.—It requires but a hasty examination of the characteristics of the harbor of New York, to prove that its natural advantages are unrivalled—perhaps unequalled—in the world; it possesses every feature desirable in a great commercial port. The city is situated at the mouth of a great river, navigable for nearly 160 miles by vessels of large size, and forming with its connecting canals and the lakes, a broad highway to the far West. Its bay is expansive, and land-locked by two great natural breakwaters, Long Island and Staten Island; its waters are deep and its shores bold. It has two broad outlets to the ocean, by Sandy-Hook and by Hell-Gate and the Sound. The city has 25 miles of water front, within the limits of the island, and this has lately been largely increased by the acquisition of territory north of the Harlem River; all of which is available for quay and wharf purposes. The upper bay includes within its area 13 square miles of safe anchorage for large vessels, and the lower bay about 88 square miles. The average rise and fall of its tides is less than 5 feet. (See Appendix I).

PHYSICAL FEATURES OF THE ISLAND.—The lower part of the island is generally formed of sand and gravel overlying granitic rock. The upper portion, which is high on the west side, is generally rocky, the rock being granite, gneiss, mica-schist and blue-bolders, of a quality fit only for ordinary foundations. A considerable depth of black and blue mud overlies the sand, gravel or rock of its shores. Figs. 1 to 8, showing boring-sections made by Col. Mechan and Maj. Watson of the Department of Docks, give in detail the character of the bottom. At and

Rock Borings North River.

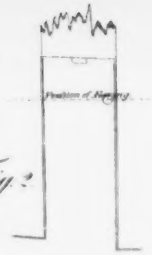


Fig 2

Fig 4.

WEST 13TH ST.

PIER Nº 60.

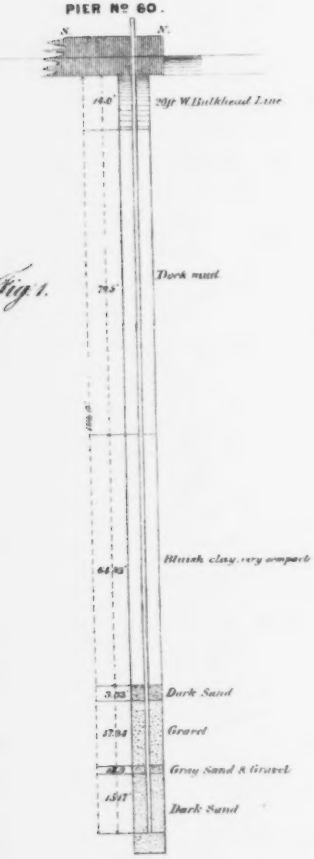


Fig 1.

CANAL ST.

PIER Nº 42.

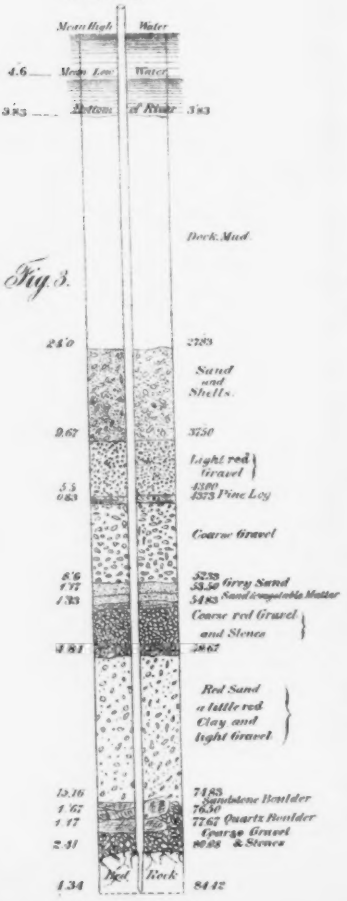


Fig 3.

Penetration 4.34.

From M.L.W. to Red Rock 80.08

BATTERY PL.

PIER Nº 1.

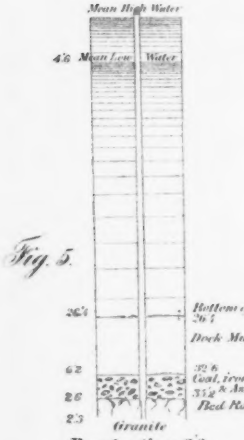


Fig 5.

Penetration 2.3

From Mean Low Water to Red Rock

WHITEHALL ST.

PIER Nº 1.

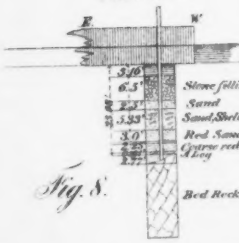


Fig 8.

East River.

PIKE ST.

PIER Nº 41.

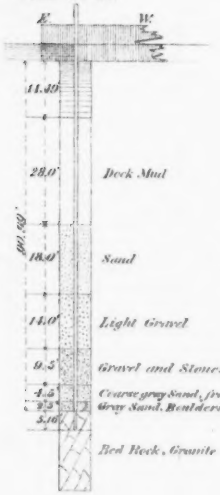


Fig 6

MAIDEN LANE.

PIER Nº 18.

Fig 7

above Eleventh Street, on the North River, the stratum of mud rapidly deepens, until in some places, it reaches a depth of over 100 feet, varying in its consistency from that of semi-fluid material to that of stiff blue clay. On the East River, hard bottom is reached at more moderate depths. Good holding ground for piles obtains, with few exceptions, along the street line as now established from West Eleventh Street to Corlear's Hook.

The climate, except in extreme winter, is genial, and the surrounding country surpassingly rich and fertile. Nature has thus given to New York all the natural advantages possible in a commercial port, and no expensive or difficult problems of construction stand in the way of making its water-front the most available one in the world.

Historical Notices.—The necessities of the past have established and developed the proper general system of docks or wharves for New York, without the intervention of any peculiar engineering skill. A glance at the history of its development will conclusively prove this.

New York Island was discovered in 1609, by Henry Hudson, sailing in the "Half-Moon," in the employ of the Dutch East India Company, and the first regular settlement upon it was a trading post established at its southern extremity by Christiaanse, in 1615. "The earliest record connected with wharves and docks in the city, is one dated 1654, when Daniel Litchoe, tavern keeper, was authorized to build a dock on the Strand." (*Hoffman on the Corp.*)

In 1656, the settlement had increased to 2,000 souls, and a brisk trade in fur and tobacco had been established with Europe. A map published in that year by Dr. Adrian Van der Donks, shows at least one wharf, near the present site of the Battery (perhaps the "dock" built by Litchoe). "There was an ordinance passed in 1660, which recited that the Burgomasters had received permission from the Director General to take a certain sum from shippers and owners for the erection of a pier for the accommodation of the inhabitants." (*Ibid.*)

In 1672, the province of New York owned 6 ships of 100 tons burthen, trading with Europe; and in 1676 a new dock was built and paid for, by a tax levied on 301 merchants. In 1696, the merchant marine amounted to 60 ships, 40 "boats," and 62 sloops, and the houses numbered 594. In the Dongan Charter (1686) we find mention of "the "bridge into the dock, the wharves or docks and their appurtenances."

On the map of 1695 (the oldest on record here), there appears an enclosed dock just east of Whitehall Street, with a bridge or pier running

into it (undoubtedly the one referred to in the charter of 1686), and another at the foot of Wall street, marked as the "new dock." It appears then that so far back as 1686, the city had a dock system established. The assessed valuation of city property was then about £75,694.

In 1728, the dock system, as shown by a map of an actual survey made by James Lyne, consisted of alternate narrow slips and broad "keys" or moles, only 4 jetties or wharves appearing (on the North River). This system extended up the shore of the East River from the Battery, for nearly one mile, and furnished a very short wharf-line for the extent of water frontage occupied, on account of the great distance between the slips.

By the year 1782, as appears from a map by I. Hills, the dock system had been entirely changed. A largely increased, active and localized commerce, requiring a greatly extended wharf-line within a limited extent of water front, necessitated the adoption of the present system of narrow wharves and slips, by which the largest available wharf-line can be obtained with the shortest water frontage. The keys or moles had already been built upon, and the wharves thrown out just as now. The system had also been developed along the Hudson River for a considerable distance. The process of filling out into the two rivers had also been commenced and had rapidly progressed. To give an idea of the extent to which this has been carried since the early days of the city, it may be remarked that in 1687, Water Street, on the East River had not yet been built; and its inner line coincided with the low water-mark of that date, just east of Whitehall Street. In 1739, high water-mark on the Hudson River was 9 feet east of the easterly line of the present Greenwich Street at its southerly end.

The plan of construction of the early wharves, where hard bottom could be reached at ordinary depths, consisted in alternate cribs of wood filled with stone, and bridge-ways of from 10 to 20 feet span. Where holding ground for piles could be found, undoubtedly piles were in many cases used. The retaining walls or "bulk-heads" were constructed of cribs, as now, but the carpentry was of the rudest kind.

The ordinances of 1795 and 1796, and the Legislative Acts of 1798, and of 1813, amending the same, established South and West Streets as the exterior limits of the city along the rivers, giving a width of 70 feet to each of these streets. By the above mentioned Acts, the city authorities were empowered to alter and extend along the shore these streets as

they deemed best, so as to accommodate them to the sinuosities of the shore lines and the growth of the city. These two Acts were still in force in 1853. By them authority was given to the city authorities to build wharves, bulk-heads, &c., and to alter them as they deemed best. Numerous supplementary Acts were also passed regulating the lines of the avenues along the northern portion of the city.

Under the Act of 1855, a board of Harbor Commissioners was appointed by the then Governor, to establish the bulk-head and pier-head lines of all the shores belonging to the city. The lines established by them are known as the Harbor Commissioners Lines of 1857, and were the legal lines until those of the Department of Docks were adopted in 1871.

For many years (indeed from the earliest day) the management of the wharves and slips belonged to the bureau, or department of the city government, having control over the streets. For want of adequate appropriations and proper care, they were generally in a delapidated condition; unsafe, unsightly and ill-defined. Many of them belonged to private parties, grantees of the corporation, who often did not receive sufficient revenue from wharfage to pay for repairs.

In 1867, under the Mayoralty of John T. Hoffman, the Commissioners of the Sinking Fund appointed a Board of Examiners to estimate the value of the wharf property belonging the city, and to devise a plan for its improvent. They proposed certain repairs and extensions, and reported the following schedule of valuations and costs:

Present value of wharves, piers and slips owned by the city			$15,793,500
Cost to repair the same	$1,119,185.	Value when repaired	$18,707,400
Cost of extending as proposed	$791,550.	Value when extended	$20,377,100

This estimate covered the value of the structures themselves, and did not include the value of the lots upon which they stood. The present value of these structures will, at least, reach $20,000,000.

The increasing demands of the shipping interests for better wharf accommodations, and increased facilities for tran-shipment of freight, &c., at last caused the Legislature of 1870 to organize a Department of Docks. The government of the department was vested in five commissioners. They were to determine upon a plan of permanent improvement of the water-front; and the administration of all the wharf property belonging to the city, and the regulation of the whole water-front devolved upon them.

This was the first successful step towards the adoption of a plan of

permanent improvement of the water-front. The Engineer-in-Chief appointed by this Board was Gen. George B. McClellan. The following extracts from his report gives the conclusions reached by him after a very careful study of the question—"Bearing in mind the relative tonnage of "the ports, as shown in this statement, also the extremely moderate "variations of the tide here, as well as the fact that our rivers and bays "afford excellent and spacious anchorage grounds and a natural water-"front incomparably superior to that of Liverpool or London, it is evi-"dent that there is no necessity for our resorting to the English system "of enclosed docks; it is safe to say that, without reference to its exces-"sive cost, such a system would be absolutely pernicious here. From "what has been already stated, it is clear that the arrangement best "suited to our wants is a continuous permanent river wall, so located as "to widen the river street very considerably; with ample piers project-"ing from it at sufficient intervals, and so covered as to afford security "against the weather, fire and thieves, wherever necessary. This is the "simplest, most convenient, and by far the most economical system that "can be suggested. It brings into play all the extraordinary natural ad-"vantages of the port, and it affords every facility for the cheap and "rapid handling of vessels and their cargoes."

"In brief the general system proposed is:—First. To construct a "permanent river wall of *beton* and masonry, or masonry alone, so far "outside of the existing bulk-head as to give a river-street 250 feet wide "along the North River, 200 feet wide on the East River from the Battery "to Thirty-first Street, and 175 feet wide north of that point. Second. "To build piers projecting from the river wall of ample dimensions, "adequate construction, and so far as possible, affording an unob-"structed passage for the water. Third. Whenever it is necessary, to "cover these piers with substantial sheds, suitable to the requirements "of each case.

"In carrying into execution the proposed improvements along the "water-front of New York, it is obvious that they should be extended "only as the requirements of the commerce demand. The commerce of "New York is now accommodated by the following extent of wharf faci-"lities, viz—North River from the Battery to Sixty-first Street, a bulk-"head with an aggregate length of 23,163 feet, and an aggregate length "of piers of 31,229 feet, with a pier area of 1,606,024 square feet; East "River from the Battery to Fifty-first Street, 26,494 feet of bulk-head, "and an aggregate length of piers of 19,139 feet, with a pier area of "716,644 square feet. Thus the bulk-head and piers together give a "wharf-line of 150,395 feet, or 28½ miles, with a pier area of 2,322,668 "square feet. By wharf-line is meant all that portion of the river wall "and piers that vessels can approach. Considerable portions of this "wharf-line are practically useless from insufficient depth of water and "other causes. The proposed arrangement of the water front, including

"the new river wall and a far better disposition of the piers, gives on "the North River from the Battery to Sixty-first Street, a river wall line "of 27,743 feet and a pier length of 37,529 feet, with a pier area of "3,325,600 square feet; and on the East River, from the Battery to Fifty-"first Street, a river wall line of 27,995 feet, and a pier length of about "28,000 feet, with a pier area of about 1,780,000 square feet.

"Thus, in the proposed system, the pier and river wall together will "give a wharf-line of about 195,000 feet, or about 37 miles, and the piers "alone will have an area of about 5,105,000 square feet, sufficient, it is "safe to say, to accommodate a commerce vastly greater than that which "now finds its way to the water-front of New York. It would thus ap-"pear that, for a very long time time to come, it will be necessary to "extend the new system over a portion only of the water-front between "Forty-first Street, North River and Fifty-first Street, East River."*

It will be observed that the general system adopted in 1871 is the same which the necessities of the past had already established as the proper one for New York, even as far back as 1782, viz.: a system of narrow wharves and slips, affording the longest wharf and quay-line for the shortest extent of water-frontage, combined with readiness of access. This plan was adopted by the new board of the Department of Docks in 1873, upon the recommendation of Gen. Charles K. Graham their Engineer-in-Chief, and the operations now carried on by the department are governed by it.

There can be little doubt among engineers who have studied the question, of the wisdom of adopting this general plan. In regard to the details there will probably be a diversity of opinion; this, however, is likely to be limited to the question of wharf construction,

* In order to give a general idea of the importance of the interests most nearly concerned in the development of a proper wharf system in New York, the following extracts have been taken from the last report of the Chamber of Commerce (1872-73), compiled from the official reports of the Secretary of the Treasury, and the Chief of the Bureau of Statistics on Commerce and Navigation:

At the Port of New York.

Year.	Foreign Imports.	Foreign Exports.	Domestic Exports.
1860	$233,692,341	$17,514,689	$120,630,955
1872	418,515,829	15,161,218	270,413,674

Total annual tonnage of Port of New York in 1872:

	Engaged in foreign Trade.	Engaged in coastwise Trade.
Entered	3,969,339 tons.	1,754,810 tons.
Cleared	3,748,516 "	2,331,293 "

for there can be little question that the most satisfactory plan of establishing a *permanent river street* is to limit it by a wall of masonry. No other plan is sufficiently durable, nor can be even temporarily satisfactory. Our present wooden bulk-heads are constantly sinking, sliding out, and overturning and filling up the river, and besides, owing to their situation, they rapidly decay and are unsightly. They, in a few years, cost the city more than a masonry wall.*

The River Wall recommended by Gen. McClellan, and adopted by the present department, is composed of *beton* blocks weighing from 25 to 50 tons each, extending from the foundation to within about 2 feet of low water-mark, and above this level of concrete laid *in mass*, faced with ashlar granite masonry. The section of the wall is shown Figs. 5, 9–10. The idea of using large *beton* blocks for this work is due, I believe, entirely to Gen. McClellan, and the late operations of the department show that the plan is an exceedingly expeditious and cheap one.

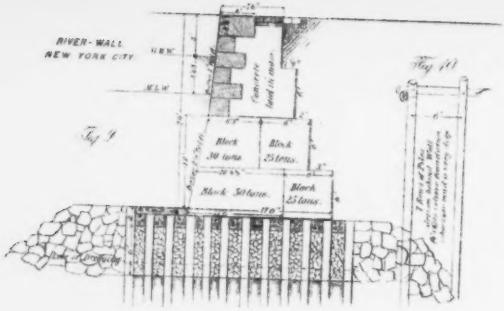
The blocks are composed of—by volume—1 part of Portland cement, 2 of sand, and 5 of stone (Bergen trap), broken to pass through a 2-inch ring. These proportions have lately been changed to 1 part of cement, 2½ of sand, and 6 of broken stone. When concrete is laid loose under water these proportions are changed respectively to 1, 2 and, 3 or 4 parts. The resulting weight of the blocks is from 150 to 152 pounds per cubic foot. The crushing strength, as determined by the writer in 1872, is after 8 months from 944 to 1,666 pounds per square inch, and after 6 days from 527 to 1,055 pounds per square inch

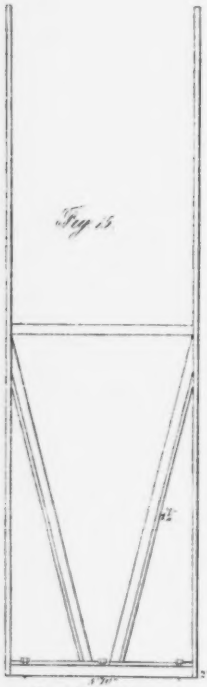
The blocks are laid by means of a floating derrick of "Bishop's" plan, designed by Mr. Isaac Newton of this city, a very complete and excellent machine—sketched, Fig. 11. The performance of the derrick on the Christopher Street section, this year, has been as follows :

23	blocks of	50	tons,	weighing	1,400 tons.
27	"	" 30	"	"	810 "
54	"	" 25	"	"	1,350 "
A total of.....					3,560 " or 1,780 cubic yards,

laid in 18 days, in 14 feet of water. The total time occupied in loading, transporting, and laying was 29 days. The maximum performance was 14 blocks, weighing from 25 to 50 tons, laid in one day of 10 hours. The total cost of loading, transporting and laying in 14 feet of

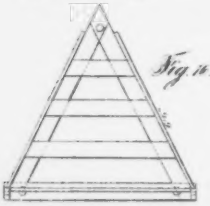
* There are cases of comparatively recently constructed bulk-heads within the limits of the city, below Fortieth Street, where these structures have slid out nearly 15 feet.





Front Elevation.

Side Elevation.

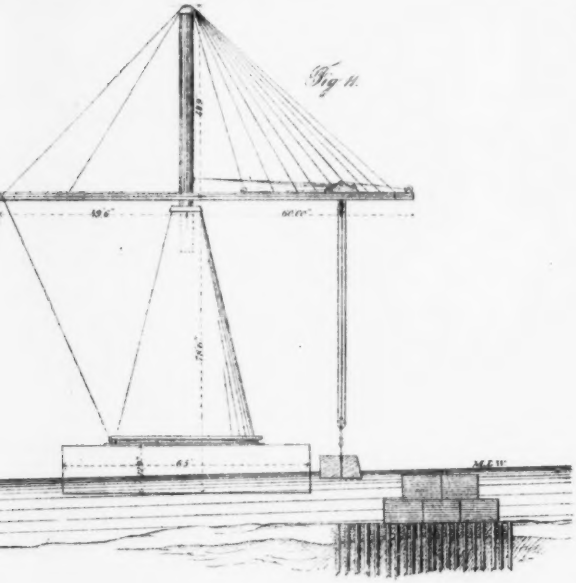


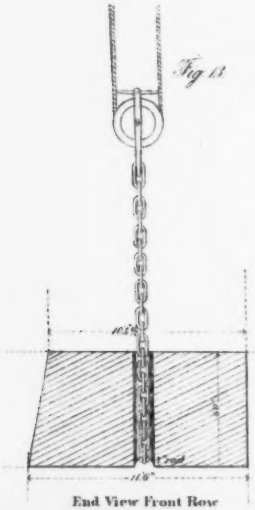
Plan.

GUIDES for laying Stone

Skeleton Sketch of Derrick, *Lowering 50 Ton Block*

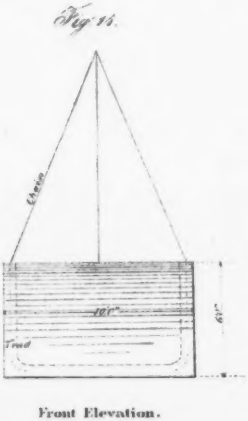
Boom athwartship.





End View Front Boye

Base Block 50 Tons



Front Elevation.

50 Tons.

Groove 7" high
" 11" wide

water, will not exceed $1.50 per cubic yard, including interest on cost of derrick, and wear and tear. The cost of the blocks, exclusive of rent of yards, is about $12.50—with cement at $5 per barrel, the material alone costs $8.50 per cubic yard. It does not cost the city, considering *all* expenses, over $16 per cubic yard *laid*.

The wall is founded, as the bottom requires, on rip-rap, or piles—generally the latter. In laying the blocks on rip-rap, the foundation is first levelled off with broken stone, and over this about one foot of concrete is laid and levelled by divers; a sweep resting on two guide-boards, laid on the extreme edges of the foundation, being used for the latter operation. The blocks are then laid by means of the derrick, a light frame of wood supporting two vertical guide-rods being placed on the top surface as the block sinks; the alignment is made altogether by means of these rods, which project above the water. Figs. 12–16. The blocks are then loaded with about twice the weight they are to bear, for some months before the superstructure is added.

The pile foundation is cut level with a circular saw as usual, and the piles are then packed round with broken stone and concrete. In this case the blocks are not weighted.

A section of about 900 feet of this wall is now in process of construction between Christopher Street and West Eleventh Street, in conjunction with the corresponding piers (which are of wood), built for the "White Star Line" of steamships; another section of it was completed at the foot of Battery Place last year. It may be remarked that on the section built there, the blocks were forced down by the weights nearly one foot, sinking, however, very evenly. In this case the bank of rip-rap was about 30 feet in height, and carefully put in with large and small stones; the bottom had previously been dredged down to rock and firm sand or gravel.*

Wharves.—The question of wharf construction is in a great measure distinct from that of the building of the river streets. These streets are built for the use of the whole community; the wharves are built for a special interest—the shipping interest. The cost of building the river streets therefore should evidently be borne by the whole community, *i. e.*, the city should pay for their construction and maintenance; while

* The writer has always contended that the *central* portions of an embankment of rip-rap, upon which masonry is to be erected, should be well rammed by a heavy weight, which can be operated with an ordinary pile-driver. At the Battery this was partially done; no noticeable settlement has taken place in this section.

on the other hand, the shipping interest should be required to pay for the construction and support of the wharves of the city which are built for its special accommodation, and a moderate rent for use of the water-front, which should be applied to the extension of the wharf system. But it could not be a sound or just policy to exact from the shipping interest *more* revenue than is required to pay for and support the special facilities afforded it. To do so, would be to introduce a system of unequal taxation, discriminating between portions of the same community; since the merchants pay also their share of general taxes, and the welfare of commerce—other things being equal—is also the welfare of all.

The problem then, presented for solution would appear to be: of the various systems of wharf construction affording equal facilities for the transhipment of freight, which will be most economical for commerce;—cheap wharves of short durability, requiring very frequent rebuilding; or more expensive ones of greater durability; and shall they be of wood, iron or stone?

The question can be more concisely stated thus: Given, the cost and lives of various wharves, all satisfying the engineering requirements of strength and stability and affording the same facilities, to determine which of these involves the least annual outlay for its maintenance. The problem can be solved by comparing the rentals which must be exacted from the various types of wharves to make them self-sustaining. The elements of rental are: 1st. Simple interest. 2d. Accumulating fund to pay off bonds issued to pay for construction. 3d. Accumulating fund to rebuild at the end of the life of the wharf. 4th. Accumulating fund to re-plank and repair. 5th. Accumulating fund to pay for loss of rent at each re-building. 6th. Cost of administration (the same in all types, and not included in the comparison). 7th. Rent of water-lots (the same in all types, and not included in the comparison).

Appendix II* gives estimates carefully made by the writer, of cost and lives for the principal types of wharf construction. The costs of the wooden and stone structures have been tested by actual practice in the city. The standard dimensions in the comparison will be taken: length 500 feet, width 60 feet, and depth to firm bottom below dock from 50 to 60 feet. Considering them in order, we shall find the rent:

*Dimensions of wharf, 500 by 60 feet=30,000 square feet of deck. Safe load=610 pounds per square foot of deck. Factors of safety—for wood—10 for beams, 5 for piles—for iron, 5 to 6. Depth of water at low tide, about 25 feet. Rise of tide about 4.63 feet. Bottom—mud and clay, overlying sand and rock. Assumed depth from deck to hard bottom, 50 to 60 feet. In each case the cheapest practicable span is taken.

No. 1.—Natural Wood—Cost, $43,000:

1st. Simple interest	$3,010 00
2d. Sinking fund to pay bonds (30 years)	456 00
3d. Accumulating fund to rebuild (15 years)	1,711 00
4th. Accumulating fund to re-plank, &c. (5 years)	870 00
5th. Accumulating fund for loss of rent at each rebuilding ($2,500 every 15 years)	100 00
	$6,147 00

No. 2.—Treated Wood—Cost, $60,000:

1st. Simple interest	$4,200 00
2d. Sinking fund to pay bonds (30 years)	636 00
3d. Accumulating fund to rebuild (35 years)	432 00
4th. Accumulating fund to re-plank, &c	471 00
5th. Accumulating fund for loss of rent at each rebuilding ($2,500 every 35 years)	18 00
	$5,757 00

No. 3.—Natural Wood—Cost, $87,000. (Best possible construction):

1st. Simple interest	$6,090 00
2d. Sinking fund to pay bonds (30 years)	922 00
3d. Accumulating fund to rebuild (15 years)	3,463 00
4th. Accumulating fund to re-plank, &c	870 00
5th. Accumulating fund for loss of rent at each rebuilding ($2,500 every 15 years)	100 00
	$11,445 00

No. 4.—Treated Wood—Cost, $106,000. (Very best possible construction):

1st. Simple interest	$7,420 00
2d. Sinking fund to pay bonds (30 years)	1,124 00
3d. Accumulating fund to rebuild (35 years)	763 00
4th. Accumulating fund to re-plank, &c	471 00
5th. Accumulating fund for loss of rent at each rebuilding ($2,500 every 35 years)	18 00
	$9,796 00

No. 5.—Iron Screw Piles—Cost, $187,000:

1st. Simple interest	$13,090 00
2d. Sinking fund to pay bonds (30 years)	1,982 00
3d. Accumulating fund to rebuild (50 years)	467 00
4th. Accumulating fund to re-plank and paint	1,471 00
5th. Accumulating fund for loss of rent ($5,000 every 50 years)	13 00
	$17,023 00

No. 6.—Cast-iron Columns filled with Concrete—Cost, $367,000:

1st. Simple interest	$25,680 00
2d. Sinking fund to pay bonds (30 years)	3,890 00
3d. Accumulating fund to rebuild (50 years)	917 00
4th. Accumulating fund to re-plank and paint	1,471 00
5th. Accumulating fund for loss of rent (50 years)	18 00
	$31,986 00

No. 7.—Masonry, Concrete and Granite—Cost, $450,000:

1st. Simple interest	$31,500 00
2d. Sinking fund to pay bonds (30 years)	5,220 00
3d. Accumulating fund for repairing, &c. (15 years)	650 00
	$37,370 00

In the above comparison, it appears that in a very large ratio, the wooden structures are more economical than the others. No. 1 is the cheapest structure that will satisfy engineering requirements, and No. 4 is the best possible wooden structure. It also appears that if the wood could be artificially preserved at the cost of the estimates (about 25 cents per cubic foot), so as to last 35 years, the structure would prove cheaper than those constructed of natural wood. The type of structure therefore which will prove most economical and satisfactory, is that lying between No. 2 and 4. The inner portion might be made of the type of No. 2, and the outer portion or pier-head made of the type of No. 4. The comparison will continually grow less favorable to the wooden structures, as labor and interest rates grow cheaper and wood becomes more costly.

Lives of Wharves.—It will be observed that a very important element in the above comparisons, is the *life* of the structure. In regard to the life of natural wood *unprotected*, it will be admitted that from 10 to 15 years is all that can be counted upon, even with the best of timber —*used as it is in wharves*. A personal inspection of the wharves of the city and records of other similar works has established this; and 15 years has been taken as the life beyond which a wharf built of unprotected natural wood will not be worth repairing. The data set forth in the report of Gen. Cram, U. S. Engineers, to the government, on the "Preservation of Wood," and other similar reports, prove that wood can be artificially preserved, so that its life will be extended beyond 30 years, even perhaps to 40 years. In fact the life of the timbers in wharves if merely oiled or painted annually, after they have become seasoned

and dry during the heat of summer, taking care to seal all joints with pitch—can certainly be doubled. Indeed there is no reason why with proper care these timbers, excepting the *deck*, should not last as long as those of houses. It would perhaps be best that the bottom faces be left uncoated for several years, so as to leave free egress for the moisture and sap.

The life of a wooden wharf will, however, depend mainly upon that of the sub-structure, the piles or columns. Creosoting, if well done, will effectually preserve them against the worms for perhaps 30 years. A process which promises success, is to construct the columns or piles of 4 square timbers, each 10 by 10 inches, or 12 by 12 inches in section, bolted and trenailed together, and to protect them above the mud-line with a clay pipe filled in with concrete; the outer columns could be protected by iron casings; a casing of cast-iron similarly filled, also furnishes an excellent and sure protection. The sea-worms (Teredo Navalis), never eat below the mud-line; nor are their ravages severe when the water is dirty or impregnated with the refuse of gas-houses and sewers. From a careful examination of this question by the writer, it appears that the life of an unprotected oak, pine or spruce pile along the shores of New York Island, is from 8 to 10 years, at points where the *water is clean*, and at least 15 years along the greater portion of the shores, and out from 100 to 200 feet into the stream. The ravages of the worms of late years have been greatly diminished by the sewerage and refuse of gas-houses flowing along the shores. There are cases where the piles have been standing along the shore for 20 to 30 years, and remain comparatively free from the worms.

Submerged bolts and other wrought-iron parts are generally badly corroded in less than 20 years; the corrosion taking place in lines parallel with the fibres. Certain kinds of cast-iron could, perhaps, be made to last 50 years; but already authentic rumors are afloat that our cast-iron light-houses are materially injured by corrosion; 50 years is a generous allowance and probably greatly exceeds the average life of cast-iron in salt-water. It is very doubtful whether any type of iron wharf will pay by its increased durability, for its greatly increased cost over that of a wooden structure. It must likewise be remarked that a wooden wharf affords the same facilities as an iron one, and is much easier for vessels.

In the opinion of the writer, it may be confidently asserted that the

time for building iron or stone wharves in this city has not yet arrived. Such wharves should, if built at all, be introduced very gradually, and only by means of surplus funds. The time may come when capital and labor will be cheap enough to make it a good investment to build iron or stone wharves in this city, but it is far distant.

The Department of Docks is now building four large wharves of wood, on the Christopher Street section, of an improved quality, and has nearly completed all of them. Another is being built at Canal Street, North River. In these structures the Department has not found it possible to allow the Engineers to introduce any artificial preservatives except external coatings of fish-oil and paint, but in the general character and strength of the wood-work and fastenings, every care has been taken to make them complete and of the very best quality. The pier-heads are the only novel features of these piers: they are constructed of built up columns, 20 by 20 inches in section and 75 feet in length, placed in rows 12½ feet apart and 9½ feet apart in the rows. The rows are sheathed from low-water up to the girders on both sides, with 5-inch planking, the ends of which are protected with boiler plates. The heads of the columns are securely framed into the caps and girders. The piles used in the pier, some of which are 95 feet long, are driven in rows 8 feet apart, 5 feet apart in the row, and securely braced. The square timber is 12 by 12 inches in section. The columns are likewise braced with 1½-inch rods, extending from the bottom to low-water.

Much has been said against wooden wharves on account of their liabilty to take fire. The same argument could be urged against our wooden shipping. It must also be observed that a wharf is so placed that there is little chance of a fire spreading to adjacent buildings: the flames being low on the water, would, to catch, have to leap over a water-space of 150 to 200 feet, or over a wide river street. The records do not show that the city has lost many wharves from fire.

Rentals. – This subject, extensive in itself, can be only briefly touched upon in this paper. A rental could be justly exacted from parties making special use of a portion of the quay or bulkhead, for such employment robs the general public of a right to its free and unobstructed use. This rental should not, however, be based upon the cost of the river street and wall, but for the reasons before stated, should be at the same rate as is charged for an equal extent of wharf-line. A moderate ground-rent might also be charged for wharves, for the use of

the water lots covered by them, and this should, if exacted, be applied to the further improvement of the water-front.

When wharves are not leased, the established wharfage charges should be based upon the amount of the rental which should be exacted if leased, considering the wharf to be fully occupied during the year. The following exhibits the revenues received for several years from wharf property belonging to the city :

YEARS.	REVENUES.	YEARS.	REVENUES.	YEARS.	REVENUES.
1865.	$272,415 61	1868.	$344,441 52	1871.	$402,119 03
1866.	219,652 37	1869.	341,563 41	1872.	423,545 25
1867.	316,983 84	1870.	358,347 33	1873.*	460,000 00

Titles to Property along the Water-Front.—There are many difficult questions involved in the acquisition of the property along the water-front, claimed by private parties. There are large extents of such property in the lower portions of the city, interspersed with the lots to which the city has an undoubted title. By the Dongan Charter (1686), and the Montgomerie Charter (1730), and several subsequent Legislative Acts, all the land from high-water to low-water mark, and the land under water to a distance of 400 feet beyond low-water mark along the whole extent of the shores of the East and North Rivers, also the land to low-water mark along the Harlem River and as far beyond as the streets extended, and jurisdiction to low-water mark on its Westchester shore (also certain portions of the shores of Long Island and New Jersey), became the property of the city with authority to extend piers from these exterior limits into the rivers. Another grant was made of submerged lands along the Hudson River of about 1,000 feet in width, and of varying extent along the East and Harlem Rivers, by the Land Commission in 1871, in accordance with the provisions of the Act of 1870 which established the Department of Docks. Any titles which interfere with those of the city are based upon grants by the Corporation. A pre-emptive privilege—the first privilege of acquisition only—is reserved to the owners of the adjacent lands or lots along the upper portions of the shores; that is, north of Corlear's Hook and Charlton Street. Unless then, some grant from the city interferes in special terms, it may well be doubted whether the city has not a full title to all the shore lands beyond high-water mark.

*Estimated.

Conclusions.—The plan of construction then, which seems to commend itself as the proper one for the improvement of the water-front of New York, is: to construct a quay wall along the river street of *béton* and granite masonry, increasing the width of the street considerably, and from this wall to throw out piers of the very best quality of woodwork, preserved against decay by all possible means.

Once establish a broad river street, and good piers built and taken care of according to a well-established plan, and it will shortly follow that all known means of facilitating the handling and transportation of freight will be introduced by the interested parties.

The first duty of the city is to establish a good and permanent highway of adequate width around the water-front; all other improvements will follow rapidly, and, indeed, could safely be left under certain general restrictions to the parties directly interested in them.

It is by no means certain that it would not be sound policy for the city to sell all its new wharves as fast as they are built, of course reserving a certain special jurisdiction over them. In this way, the capital expended upon them would be immediately returned with large increase. If there ever was a time favorable to such a policy, it is now, when the city is plunged so deeply in debt.

These public works should go slowly on, say at the rate of half a mile per annum. If in ten years the wall could be completed from Grand Street on the East River, to West Eleventh Street on the North River, its progress would be all that could be desired. The city would then possess the finest dock facilities in the world.

Mr. Ashbel Welch: Historical notes were mentioned in this paper; one fact, however, was not referred to—the original location of the commercial metropolis of America was either Amboy or Communipaw. A well known historian states that the latter was first selected as the site, because the ground was soft and it was a good place to drive piles.* Now, although Diedrich Knickerbocker may have been wrong in his reason, he was correct in his fact. The right place for the wharves of New York is on the west or New Jersey side of the Hudson river, where the railroads end.

On the average, it costs $2 per ton to take goods from the railroads at Jersey City across the river to the warehouses, pay the storage and then ship them, nearly all of which is an unnecessary expense. This

* See "History of New York," by Diedrich Knickerbocker.

induced me to recommend the purchase of Harsimus Cove—70 acres of good deep mud—at a cost of $1,000,000, where wharves are now being constructed, not for New York alone but for America, to which goods may be brought from the West, unloaded into warehouses adjoining the tracks, stored as long as desired and then put on shipboard, all without carting or handling. In a short time the Pennsylvania R.R. Co. alone, will deliver at Jersey City 1,000,000 tons of freight per year, whence, by the proposed improvements nearly $2,000,000 will be saved, a considerable item in these hard times. I am now designing a grain elevator at that point, capable of handling 100,000,000 of bushels per week and with a storage capacity of 4,000,000 bushels. At present, grain from the cars is loaded into lighters, taken to Brooklyn, stored and then returned in the same way for shipment on the European steamers, all at an extra cost of no less than $1.50 per ton which is to be saved.

There is another and important thing to be gained by locating the docks and warehouses on the New Jersey side of the river, which I will illustrate. Some years ago I was sitting with the head of a leading iron house in London, in his counting-room. A clerk came in and showed him a warehouse receipt and inspector's certificate; he nodded his head. Presently another clerk handed him a check, which he signed; then a third clerk whispered something to him, he again nodded his head. The clerk went to a shipbroker's, and returning made a report. The whole had not taken ten minutes. In explanation, the principal said: "New York is a village—you cannot do business there—I was once a whole morning engaging a ship for a cargo of rails, which we imported, and two or three days getting at and inspecting them. In the brief time you have noted, I have purchased a cargo of rails from Germany, paid for them and chartered a ship to take them to New York." This, for for lack of a warehouse system, would have taken at least a day at this port.

Now, at Communipaw of which so much fun has been made or at Harsimus Cove or Hoboken, there should be warehouses for goods and a method of inspection such, that the consignee or owner, could make sales as in London upon the warehouse receipts without the buyer ever seeing the wares—the same as the Dutch grain merchant may never have looked upon a field of wheat. This is the true system, it may be established in New York, but this island is not large enough for the purpose. The lower part of the city is the counting-house of the continent,

and there is not room for wharfage, storage and the like. But if there was, the difficulty remains that the dock and warehouse must be separated at least by a street 250 wide, thus for transhipment necessitating handling and cartage twice over, which on the other side of the river will be avoided.

Mr. J. Dutton Steele—Although I am a Pennsylvanian, I cannot see that the whole trade of the great West must pass through my own State or New Jersey. An enlarged view of the United States and its commercial channels, will show that the principal avenue of transportation is on the parallel of the Erie canal—whence for example, the Hoosac Tunnel was built to divert trade towards Boston. Much of this trade seeking New York will come down on the east side of the Hudson and to receive it, wharfs and warehouses will still be required in this city.

Mr. Francis Collingwood—From extensive observations during the past winter, I am able to confirm what Mr. Welch has said of the cost of transhipping freight at this port. The expense of handling freight at the depots here frequently reaches $1 per ton, and to move it across the river, as much more.

I wish to inquire why American cement is not used in constructing the docks; would it not stand the water?

Mr. Van Buren—I have tested several kinds of American cements without satisfactory results. In the blocks it might stand, but it would not when laid loose, which in many cases is required in the foundations. Trials, however, with American cements are still being made.

APPENDIX 1.

Principal Facts relating to the Tides of New York Harbor, taken from the Map of the U. S. Coast Survey, 1845.

Establishment of harbor at Governor's Island	8 hours 19 min
Height of mean low-water above plane of reference	0 ft. 10.3 in.
Height of mean high-water above plane of reference	5 ft. 7 in.
Mean rise and fall of tides	4 ft. 8.7 in.
Mean rise and fall of spring tides	5 ft. 8 in.
Mean rise and fall of neap tides	3 ft. 8.7 in.

	Cross Sectional Areas, referred to low-water.	Currents. Knots per hour.
Hudson River, West 16th street	15,528 sq. yds.	1.5 at ebb, 1.3 at flood.
East River, Corlear's Hook	6,574 sq. yds.	4 at ebb, 5 at flood.

APPENDIX II.

TABLE OF COSTS OF DIFFERENT TYPES OF WHARVES, BASED UPON ESTIMATES OF THE DEPARTMENT OF DOCKS, NEW YORK CITY.

Description of Structure.	Life.	Span in Feet.	Cost of Substructure. Woodwork		Cost of Substructure, per sq. foot of Deck. Woodwork		Cost of Superstructure. Woodwork		Cost of Superstructure, per sq. foot of Deck. Woodwork		Total cost of Wharf. Woodwork		Total cost of wharf per sq. foot of Deck. Woodwork	
			Not artificially preserv'd	Artificially preserved.	Not artificially preserv'd.	Artificially preserved.	Not artificially preserv'd.	Artificially preserved.	Not artificially preserv'd.	Artificially preserved.	Not artificially preserv'd.	Artificially preserved.	Not artificially preserv'd.	Artificially preserved.
Nos. 1 and 2.—Ordinary wooden pile wharf. Piles in rows, 10 feet apart and 16 in each row; capped and braced. Girders 12x14 inches, 3 to 4 feet apart. Sawed timber. Deck 3-inch plank.	Natural wood, 10 to 15 years. Preserved .. 30 to 35	10.0	$ 18,000	$ 27,000	$ 0.60	$ 0.90	$ 25,000	$ 33,000	$ 0.833	$ 1.10	$ No. 1. 43,000	$ No. 2. 60,000	$ 1.43⅓	$ 2.00
No. 3.—Wooden wharf supported by columns formed of 4 squared timbers 12x12 inches in section, bolted together, placed in rows 25 feet apart and 6 in each row, braced with 3-inch solid oak planking, in horizontal courses above low-water; and at pier head also with iron rods below water. Girders continuous in 11 rows; solid beams 36 inches deep, formed of 3 timbers each, bolted and strapped together. Deck 3-inch plank.		25.0	55,000	66,000	1.83⅓	2.20	32,000	40,000	1.06⅔	1.33⅓	No. 3. 87,000	No. 4. 106,000	2.90	3.53⅓
No. 4.—Wharf supported by cast-iron screw piles, 15 inches in diameter, one inch thick, and 50 feet in length, placed in rows 15½ feet apart, and 6 in each row—192 piles. Girders, rolled iron beams; with 5-inch plank deck, paved with creosoted wooden pavement on top of plank.	Not over 50 years. Preserved	15.5	102,000	.....	3.40	.....	70,000	85,000	2.33⅓	2.83⅓	Not paved. (172,000)	187,000	5.73⅓	6.23⅓
No. 5.—Wharf supported by piers formed of concrete blocks, faced with granite above the ice line. Girders of plate iron 4 feet deep and 49.6 feet span. 3-inch plank with creosoted wooden pavement.	Superstructure not over 50 years.	49.6	162,000	.....	5.40	.....	.....	211,000	.....	7.03⅓	.....	373,000	.....	12.43⅓
No. 6.—Wharf supported by cast-iron columns filled with concrete, sunk by pneumatic method, placed in rows 41½ feet apart and 3 in each row. Diameter of column 6 feet; length of column 50 feet; thickness of metal 1¼ inches. Girders of plate-iron; 5-inch plank deck and creosoted wooden pavement.		41.5	164,000	.....	5.46⅔	.....	.....	203,000	.....	6.76⅔	.....	367,000	.....	12.23⅓
No. 7.—Wharf of stone arches, and concrete, paved with wood or stone.	1,000 years.	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	450,000	.....	15.00

XCVII.

THE UTICA LIFT DRAW-BRIDGE.

A Paper by Squire Whipple, C. E., Honorary Member of the Society.

Read June 10, 1874.

In 1871, the Legislature of New York passed an act authorizing the construction of a swing-bridge over the Erie Canal on Hotel street in Utica, for the purpose of enabling heavy loads to cross the canal without encountering the steep grades by which the stationary bridges of the city are necessarily approached. But the canal being only 59 feet wide at that point, a pier in the centre was inadmissible, while a bridge mounted on a pier upon the shore, would be so long, and occupy so much room when open, that it could not be constructed and used without the abatement of one or more valuable business stands. This would have involved an amount of expense, which neither the state authorities, nor the citizens to be benefited by the bridge, were willing to incur.

In this condition of the case, the writer was consulted as to the practicability of devising a plan of draw-bridge which could be operated by lifting up instead of swinging horizontally, whereby the anticipated benefits could be enjoyed without incurring so much expense, and occupying so much valuable space. The idea at once struck me as highly feasible, and the suggestion was answered accordingly.

In pursuance therewith, the plan of a "lift draw-bridge" was arranged, for which letters patent were issued about two years ago. The plan being thought favorably of by parties concerned, an appropriation was made in 1873 for the construction of such a bridge at the locality above named, and a contract entered into therefor. The bridge was erected during the last winter, it was put in operation on the opening of navigation for the present season, and is now working in a successful and satisfactory manner.

The plan and principles of the structure may be briefly described as follows :

The work consists of a stationary truss-bridge spanning both the water-way and towing-path, supported by 4 corner piers and towers, sufficiently high for the suspension of a movable floor or platform under the lower truss-chords and to permit the passage of canal-boats underneath the platform—together with other parts about to be named and described.

The movable platform, extending from the berm bank to the inner edge of the towing path, is suspended by iron suspension rods (one at each end of each transverse floor-beam) passing up through the cast-iron connecting-pins of truss-chords and inside of the hollow truss-posts, being connected at the upper ends with wire ropes passing over large sheaves or pulley-wheels, and connecting with counterpoise weights to balance the weight of the platform. These sheaves are made fast to line-shafts, suspended just below and inside of the upper truss-chords on either side of the bridge, so as to allow a segment of the sheave to enter a slot on the inside of the post and receive the wire rope in its groove at the centre.

The two line shafts have each a bevel gear-wheel near the centre and at opposite points, into which work pinions mounted on the ends of a transverse shaft that serves to connect the two and secure their uniform rotation, as well as that of the sheaves mounted thereon.

The platform being thus mounted and counterpoised, is lowered to the grade of the street for the passage of land vehicles and raised for the passage of boats underneath, by application to the transverse shaft of power sufficient to overcome the friction of the working parts, with a greater or less surplus to act as an accelerating force.

The shafts upon which the sheaves and gearing are hung are of Bessemer steel 2¼ inches in diameter, with journals (as large as the bars would turn) running in composition half-boxes, and caps to hold waste or cloth for retaining oil and to keep the journals properly lubricated.

The sheaves carrying the wire ropes are 3 feet in diameter, and the ropes ⅞th inch (except end ones, which are ¾th), with 6 strands of 19 wires each and a hemp strand in the centre. They sustain a tension of 1,500 to 2,000 pounds each.

The suspension rods carrying the platform, have caps at the upper ends which come to a bearing upon the connecting blocks at the feet of the truss-posts when the platform is down, and sustain the weight of

passing loads without additional strain upon the ropes or upon the sheaves and line-shafts.

The counterpoise weights consist of 12 cast-iron boxes (6 upon each side), 9 inches square, with length corresponding to the length of truss-panels, weighing about 800 pounds each, and containing about a like amount of pig-iron. These weights have a horizontal position, thus forming a continuous chain or series of weights extending the whole length of the platform on each side, individual weights meeting one another opposite the truss-posts, and each wire rope connecting with contiguous ends of two weights, excepting, of course, those at the ends of the series.

It was estimated that the platform, (60 feet long and 18 feet wide) would weigh 10 or 11 tons, making with counterpoise, 20 or 22 tons to be moved in shifting the draw, with a friction resistance equal to 30 to 40 pounds to the ton according to the condition of the working parts. Hence, there would be required an amount of power equal to 7,000 to 9,000 foot pounds to effect a movement of $11\frac{1}{2}$ feet up or down as required in the case in hand. This being regarded as equal to the ordinary labor of a man for 2 to 3 minutes, it seemed that 4 or 5 minutes, at least, would be occupied in each up and down movement. And, as a boat would pass once in 10 or 15 minutes on an average, nearly half the time would be occupied in the movements, which was considered highly objectionable. It was therefore decided to have the draw operated by a power weight, to be wound up at leisure and held in readiness when required, for moving the draw in a short space of time.

For winding up, the plan adopted was a tread-wheel, 9 feet in diameter upon a vertical shaft, with a bevel pinion working into gear segments attached to the winding-drums, which run loose upon the transverse shaft in winding, and in running down operate by means of a dog or catch and a ratchet wheel made fast to the shaft, essentially in the manner of the working of a clock propelled by a weight. The first design was to use a single weight, winding it in opposite directions to produce the opposite movements. But it was finally decided to have two weights, one to raise and the other to lower the draw. These could be both up at the same time, and the two movements made in quick succession, when desired.

The operating weights were designed to have about twice the power required to overcome friction (the one half to act as an accelerating force), and to run down at half-movement of the platform, the latter

half-movement to be effected by the acquired momentum of the platform and counterpoise; in the meantime, the friction acting as a brake to gradually destroy the impetus, and bring the apparatus to a stop without severe concussive shock.

It was roughly estimated, that having a force of about one pound to each 50 or 60 pounds to be put in motion, a space of about 3 inches would be described by the platform in the first second of time, 9 inches in the second, 15 inches in the third, &c., and thus the half-movement, about 6 feet, would require a little less than 5 seconds, with a maximum acquired velocity at mid-movement, of about $2\frac{1}{2}$ feet in a second, while the friction, being about equal to the accelerating force, would retard the motion during the last half-movement, giving equal velocities at equal intervals from the beginning and end of the movement. Thus the whole movement would be effected in about 10 seconds, which is as short a time as was deemed desirable for the purpose.

The result expected then, was a draw-bridge allowing sufficient headroom for canal-boats, taking up no more space than a stationary bridge, and which could be operated by the moderate labor of a single man during about 2 minutes for each movement, and which would not occupy more than about 10 seconds in performing the actual movement. The practical demonstration was that while the time occupied in the movement is nearly as estimated, being about 8 or 9 instead of 10 seconds; the labor of winding, instead of 2 or 3 minutes requires less than one minute, and with a slight degree of active exertion may probably be performed in $\frac{3}{4}$ths of a minute.

It becomes therefore, a question (which can be better answered after further experience), whether it were not better to dispense with the power weights and move the draw by direct application of power, thus reducing cost of construction, simplifying the process of operating the draw, and diminishing the liability to mistakes by the operator, even at the cost of a trifling loss of available time in the use of the bridge? The difference between the actual and estimated time of winding up the operating weights, results partly from the fact that the platform is lighter, in consequence of spruce floor plank instead of hard wood having been used, whence the mass to be moved is several tons less than estimated; but chiefly from an under estimate of the force a man is able to exert for a short time, as compared with his ordinary continued labor.

In conclusion, I will only remark that this plan of draw-bridge (applicable to railroad or common street purposes) is, of course, solely

adapted to canal navigation, where boats require a head-room of 12 or 14 feet in height only; and for such use it seems clearly to possess the following advantages over other plans:

1. It does not obstruct the navigable channel like a swing-bridge on a pier in the centre of the canal.

2. It occupies much less space than a swing-bridge mounted upon a shore pier, and is less seriously affected by the action of wind.

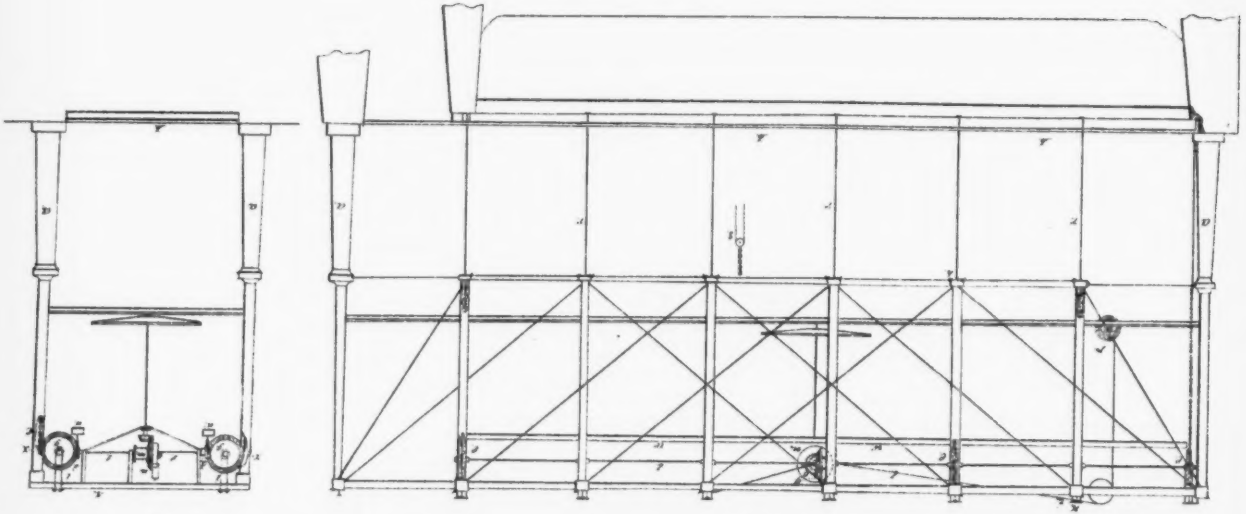
3. It is considerably less expensive in construction; and

4. It is more easily managed and worked than any other canal draw-bridge, except possibly the swing-bridge, with centre pier and narrow boat-channels on either side.

Mr. Ashbel Welch—The writer in this paper mentions, as an alternative design, a swing-bridge with pier in the canal, for which there was not room. I wish to say that the pier could as well have been on one side, and the bridge extended all the way across the canal. A large number of bridges on this plan, 42 feet long, are in operation in the State of New Jersey. There is, however, no reason why such could not be as well made 60 feet long; nor do they require much room, for when swung open as the bridge may be used for part of the towpath.

Note.—The following are references to the Plate: Fig. 1, side view; Fig. 2, end view; *a*, *a*, posts; *b*, *b*, movable platform; *c*, *c*, chains; *d*, pinion; *e*, *e*, pulleys; *f*, transverse shaft; *g*, *g* and *n*, gear wheels; *i*, *i*, hangers; *k*, *k*, cross beams; *l*, *l*, counter shafts; *m*, winding drum; *o*, ratchet wheel; *p*, power weight; *r*, *r*, rods; *w*, *w*, weights, and *x*, *x*, openings in posts.

Errata.—In "LXXXVI, Retaining Walls, an Attempt to reconcile Theory with Practice," on page 72, first column of table, for "280, 300, &c.," read "28 , 30 , &c."; and on page 73, twelfth line from top, for "that," read "thrust"; on same page, text from "By ordinary statics" to "actually required," should follow "calculated height E," on preceding page.



AMERICAN SOCIETY OF CIVIL ENGINEERS.

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XCVIII.

UPRIGHT ARCHED BRIDGES.

A Paper by JAMES B. EADS, C. E., Member of the Society.

READ JUNE 10, 1874.

In a report on the Illinois and St. Louis Bridge, in 1868, I advanced the proposition that, for railroad purposes, an upright arched bridge could be more cheaply constructed than was possible by the suspension system. This postulate called forth soon after, some dissenting views from engineering journals to which I had not, at the time, the leisure to reply.

In this paper I will endeavor to show that upright arched bridges can be more economically constructed than is possible by any other method whatever, no matter what length of span may be required. As the cost of the span (excepting the cost of roadway) will, by any system, increase at least as rapidly as the square of the distance spanned, the length of span will be limited in all cases, by financial difficulties, before grave engineering ones are encountered.

I will first point out some of the disadvantages of the system adopted for the superstructure of the St. Louis Bridge, which was the best method of arch construction then known to me, and will respectfully venture to indicate such improvements as have suggested themselves, in consequence of these difficulties being so forcibly thrust upon my attention in the progress of that work. An earnest wish to make these and their remedies thoroughly understood, will I trust, excuse in me much that may seem commonplace or didactic.

In the St. Louis Bridge the upper and lower members, which constitute a single rib of the span, are tubular in form, the tubes being each 18 inches outside diameter. The rib thus formed is 12 feet deep from centre to centre of the tubes, and the two lines of tubing are braced together by a single triangular system of vertical bracing, formed of flat bars secured to the tubes at points about 12 feet distant from each other throughout the length of the rib, the bars being placed in pairs, one on each side of the tubes, and secured to them by pins passing through the ends of the bars and the couplings of the tubes. An upper and lower line of tubes thus braced constitutes one rib of an arch or span. The curve selected for the ribs was, for greater convenience of manufacture, the segment of a circle. Each individual tube is straight, the curvature of the rib being accomplished at the junction of these individual members, each of which is about 12 feet long. The curve of the rib differs from a parabola but a few inches.

If the effects of temperature could be avoided, and the curve were a parabola, an equally distributed load on a rib would, of course, be borne by the upper and lower tubes equally, that is, half on each; and the only strain on the bracing would be in transferring half of the imposed load to the lower tube. In this case the tubes could be together and the bracing dispensed with. We should then have, in the two lines of tubes, the least possible section requisite to sustain this equally distributed load. To brace the arch against unequal loading, the tubes were placed 12 feet asunder.

The medium temperature was assumed at 60° Fah. The effect of temperature (ranging from — 20° to + 140° Fah.) increases the length of the rib about 6 inches. This extension causes the crown to rise, which relieves the lower tube of compression at the abutments, and hence that tube does not then support any portion of the weight of the rib or the load. These are borne wholly by the upper tube at the abutments, and hence its section there must be increased accordingly. At the crown, the strains are likewise changed. There, however, the *lower* tube does all the duty, as the upward bending of the rib, relaxes the compression in the upper tube at this point, hence the lower one must be increased at the centre of the rib to enable it to do this double service. It is thus seen that the upper tube at the abutments, and the lower one at the crown, must, when the temperature is at its maximum, perform the whole duty of sustaining the rib and its load. The strains in the respective tubes decrease from these points toward the

haunches of the arch, where the centre of pressure passes through the middle of the rib. At the haunches the compression is borne in nearly equal portions by the upper and lower tubes. The extra section required by temperature, therefore, becomes smaller in the tubes as we leave the crown and abutments.

These are the effects of expansion. As the ribs are formed for a medium temperature, contraction by cold reverses these strains. The crown of the arch falls, and the *upper* tube at that point must be reinforced, because it must then sustain the entire compressive strain, which, at medium temperature, is equally borne by both; while, at the abutments, the *lower* tubes have now to carry all the load. Thus it is seen that, both at the abutments and the crown, the upper and lower tubes must be each of greatly increased section on account of the changes of temperature.

These changes, in very extreme cases, will probably tend to withdraw the one tube or the other from the abutments, as extreme heat or cold affects the rib. At the suggestion of my chief assistant, Col. Flad, to give greater resistance to a change of form of the rib under unequal loading, the skewbacks in which the tubes are fastened are rigidly held to the abutments by anchor bolts. Thus, at times, a tube at the abutments may not only be relieved of all compression, but may bear tension and pull upon its skewbacks, by which an additional compressive strain may be transferred through the braces to the other tube. For this contingency additional section had to be provided at the abutments in the first two or three tubes.

Greater depth of the rib would have increased all these strains of temperature, and would thus have involved larger sections at the crown and abutments. By lessening the depth, the strains would have been diminished, but a more flexible rib, under unequal loading, would have resulted. Between these evils on either hand, after various calculations, the most economic and satisfactory results were obtained with 12 feet depth of rib.

A diagram of the strains of the central arch, on which my remarks are based, is appended, (Fig. 25). From these it will be seen that effects of temperature have compelled the use of a much larger quantity of metal in the ribs, than would have been necessary if these effects could have been avoided.

No one will, I think, deny that the cheapest possible form in which steel or iron can be used to span an opening and support an *equally* dis-

tributed load, is the catenary or suspended arch. It is equally evident that if an upright arch has the same span, weight and curve of any given catenary, the strains in the one form would be no more intense than in the other. In the upright arch they would be compressive strains, and in the suspended arch they would be tensile ones. In the suspended form, the section of metal may be solid; in the upright, it must be hollow, tubular, or of some construction that will resist buckling; but to be of equal strength it must not necessarily be more weighty. If either form have an equally distributed load placed on it, no bracing will be requisite to preserve its normal curvature. The same load, if within the elastic limit of the metal, can be borne by either with the same amount of material. Hence, the only difference in material required for the two systems for any given span and load, must lie in the difference of bracing them to resist distortion when *unequally* loaded. I refer here simply to the span between the supporting towers in the one case and the abutments in the other. Between these points, with the same curvature, the sectional areas of each may be the same, under equal loads. In this I, of course, assume that the iron or steel can be trusted with equal strains in tension or compression. In compression, tubular or other sections are quite capable of sustaining, in wrought-iron or steel, all that it is safe to apply in tension if the lengths of the compressive members do not exceed 8 or 10 diameters. In the St. Louis Bridge however, they are but 8 diameters.

The section of arch is doubled by doubling the span, and in the suspension cable it is evident that a long span involves equal sectional areas also in the shore spans, or portions between the towers and anchorages. Whereas, when the stream is spanned by a long arch, the need of such extra section in the shore spans does not exist, because they can almost always be short spans. Mr. Peter W. Barlow, C. E., in 1860, published a pamphlet on the Niagara Railway Suspension Bridge, from which I quote the following remarkably clear explanation of the fact that in all known systems of girders, with any given depth, *four times* as much metal is required to obtain the same rigidity as is needed with the arch or the cable.

"Let *A C B*, Fig. 1, represent an arch supported on abutments *A* and *B*, and let the deflection produced by a given weight, loaded equally, be represented by unity. Now, let us consider the effect of making this arch into a self-supporting structure, or bowstring girder, by removing the abutments and substituting a tie, *A B*, Fig. 2.

"Assuming the same weight, w, to be placed equally all over, the deflection will be 2, the points A and B being no longer rigid, because the tie $A\ B$ will extend as much as the arch $A\ C\ B$ will compress. Therefore, to produce the same rigidity in a bow string girder four times the metal is required as compared with the arch. The same result arises in a cable $A\ C\ B$, suspended from the two fixed points $A\ B$, Fig. 3.

"If the back chains are removed, and a compression tube $A\ B$ substituted, the metal is doubled and you have a structure with only half the rigidity. The Chepstow Bridge on the South Wales Railway is an example of this arrangement.

"The mechanical combination in the Saltash Bridge is represented by substituting the arch $A\ D\ B$ for the tie $A\ B$, Fig. 4, forming a combination of a suspension chain and an arch. The arch $A\ D\ C$ will not perform the duty of compression unless it is connected with the chains by the ties $a\ a$, $a\ a$. When thus connected both the cables and the arch assist in supporting the weight of the load. The points $A\ B$ now become fixed points, and as both the arch and the chains assist in supporting the weight, the deflection will only be half that of the simple suspension cable with double the weight of metal. It therefore appears:

"1. To convert an arch supported on two fixed abutments into a bowstring girder, four times the metal is also required to support the same weight with the same deflection.

"2. To convert a cable suspended from two fixed points into a Chepstow girder four times the metal is required to support the same weight with the same deflection.

"3. To convert the same cable into a Saltash combination (which consists of a bowstring and Chepstow girder combined, so that the horizontal tie in one case neutralizes the compression tube in the other, by which they are both avoided), the deflection is reduced one-half, with double the weight of material, or the same weight of material will produce the same deflection with the same load, as in the case of the simple arch or cable. But this is obtained at the expense of double the depth; and if the arch or suspension cable was of the same depth as the Saltash, only one quarter of the metal would produce the same stiffness.

"In the preceding illustration, the bowstring and Saltash girders are referred to—parallel girders are more commonly used—but they present no economy over the simple bowstring, and however perfect their arrangement and proportions, they will still require not less than four times the metal of a simple cable of the same depth and span to produce the same deflection."

Enough is shown by Mr. Barlow in these extracts to prove a far greater economy in the metallic arch or catenary than is possible with any truss system known. In practice, however, the chief causes which have conspired to lessen this evident economy are: 1st, the bracing requisite to preserve the form of the arch when unequally loaded; 2d,

the effects of temperature, and 3d, the necessity of heavier masonry to resist the pull of the cables or the thrust of the arch.

I cannot better illustrate the notably small amount of bracing or "girder-power" required to resist the effects of unequal loading in the suspended arch or cable than by again quoting from the same distinguished author. Mr. Barlow says:

"If the beam *A B*, Fig. 5, be divided into two beams by being supported at *C*, the two half beams, *A C* and *B C*, will deflect one-eighth of the amount of the entire beam *A B* with the same weight. Let us assume this to be a girder attached to a chain, and a load placed at *D*, the effect will be to distort it into the shape shown in Fig. 6. The deflection by the weight at *D* will cause a corresponding elevation at the point *E*, and the girder will assume the shape represented by dotted lines in the figure, to produce which a force equal to double that for a given deflection on half the beam is required, from which it is evident that the wave produced by a given weight at *D* will only amount to one-sixteenth of the deflection the same weight will produce on the entire beam resting on its two ends.

"In the above proposition it is assumed that the beam is supported at its centre point only; in practice, when attached to a suspension cable, it is supported at various points of its length, the difference between the wave of a supported girder and the deflection of an unsupported girder will therefore be greater than one-sixteenth."

"In order to arrive at the result by experiment, I had a model of the proposed Londonderry bridge on a scale $\frac{1}{48}$ of the actual span, the length being 13 feet 6 inches between the bearings, a length exceeding that of the average of the models used by the Iron Commissioners in their experiments, and is amply sufficient—due allowance being made for the scale—to determine the accuracy of the deflections on the actual girder. The principal object of these experiments was to ascertain the deflection of the wave of a girder attached to a chain, as compared with the deflection of the same girder detached, which being obtained, it is perfectly easy to arrive at the deflection of the wave of any required suspension girder, because we have sufficient experiments on actual girders of various dimensions to obtain the deflection from a given load on the same girder not attached to a chain. These experiments gave a mean result of $\frac{1}{25}$, so that, it being first determined what amount of deflection is to be the limit with a given load in a given bridge, you have only to arrive by calculation at the sections of metal of a girder of the same depth which would deflect 25 times that amount."

The economy of a continuous girder, resting on a central support, depends upon the several supporting points remaining absolutely stable, or in line. The sinking of one or the elevation of another, creates extra strains in the girder which may destroy all the advantages of continuity. The great

economy of girder power theoretically shown by Mr. Barlow is not attained in any suspension bridge of which I have knowledge, because the effects of temperature have not been avoided in their construction. The extension and contraction of the cables cause the central part of the girder to fall and rise, whilst the ends of the girder remain at a constant level. This is not avoided even where the towers are of iron also. Additional strains are thus created in the girder, for which increased sections are required. These strains can be avoided however, by jointing the cables at the centre of the span, and cutting the girder in two at the same point. By this means each half length of girder trusses the half length of cable to which it is attached. If the end of each half girder be securely fastened to the central joint, the economy of a continuous girder may be attained, because the effect of the load in bending the one half girder down is resisted by the other, just as it would be if the two were continuous. The loaded one cannot deflect without distorting the cable above it, and the cable cannot deflect, without rising where it is unloaded, which movement could only occur by bending up the unloaded half girder beneath it. In this case the cable must be likewise jointed at the towers, as each half of its length is rendered rigid by its connection with the half girder, and the half girders must also be allowed to expand and contract at the towers.

Fig. 7 illustrates this method of eliminating the strains of temperature from the suspension bridge, and securing the fullest benefit of "girder power" with the greatest economy attainable by this system of bracing. If Fig. 7 be reversed, as in Fig. 8, the dotted lines will show that exactly the same economy of bracing or "girder power" is attainable in the upright arch by this method, except that in one case the vertical connections act as *ties*, and in the other as *struts*, and inasmuch as the compressive strains on the long struts will necessitate greater sections than the tension strains require in the ties, a slight advantage is found in the suspended arch. Owing to the weight of the cables, this advantage will be increased by a difference in the expense of wind bracing. In this last item, the difference will be found to be less, however, than is generally supposed.

This fact may be thus illustrated; if a column of metal be suspended by its upper end, its weight will resist the deflection caused by a current of wind that would blow it over, if standing on its base. It must be remembered, however, that if suspended, the resistance of the weight is only effective *after* the centre of gravity

of the column has been moved. When vertical, its resistance is *nil;* the maximum effect being exerted when the horizontal position is reached by the column. Therefore to preserve the lower end absolutely from movement by the wind, would require as strong a brace or guy as would be needed to preserve the same stability at the upper end if the column were standing. The case is identically the same in the suspended and upright arches. The bridge should be braced against all movement by wind as far as possible, and hence the stability of the cables due to their weight, will not, on investigation, be found to possess as much importance as has been claimed for it. The plan of spreading the arch at its bases, as proposed by Telford in his 600 feet span for the Thames, has been applied in its reverse form to the suspension bridge, as a means of securing greater stability against wind pressure. Either application of this feature, suspended or upright, involves the necessity of increased section in the cable or arch, the supporting power of either being lessened as the plane of the curve inclines from the vertical. The increased section therefore involved in this means of securing stability should be charged in the estimate of wind bracing.

The illustration I have given of the suspended and standing column, may likewise be referred to here, to show that for rigid structures, the weight of the cables in preserving the suspension bridge from a change of form under unequal loading is not an element of so much importance as advocates of that system claim. Let *A B*, Fig. 9, be a suspended arch loaded at *D*, and *C* be the point to which the greatest curvature has been removed by the load from the centre *E*, of the span where it was when the cable was in equilibrium. By reversing the figure, we see at once that in the upright arch, the flattening from *A* to *C*, increases the thrust of that portion against the remainder of the arch, which being without load, and of inferior weight, can only be sustained by the strength of its bracing or girder power. This change of form in the arch lessens its ability to resist the effect of the load, and even increases the power of the weight, to further distort it by the additional horizontal pressure against the portion *C B*, due to the flattening of *A C*, whilst the curvature of *C B* becomes less favorable for resisting this horizontal pressure. In the cable, these conditions are exactly reversed. The effect of gravity upon the flattened portion tends to resist a further distortion of the catenary. The tensile strains in this portion increase with the straightening between *A* and *C*, and thus equilibrate the effects of the load at *D*.

If we apply the illustration of the column in explaining these phenomena, we see that if a force shall have *already* deflected the suspended column, the application of a load to its lower end will tend to restore its vertical position, and will lessen the strain on any horizontal brace or guy employed to resist the deflecting force ; but if the column be placed on end and deflected, the imposition of an additional load upon its upper end will tend to overthrow it, and will increase the strain on the brace or guy, resisting the deflection. These effects are precisely the same with the arch, when suspended and upright.

If we desire to prevent any appreciable deflection of the suspended column from its normal or vertical position, it is evident that we must apply the same amount of horizontal bracing to resist the deflecting force that would be required to preserve the column vertically against the same force if the column were standing on its end, for so long as the suspended column remains vertical, the resistance due to its weight or that of the load is *nil*, as before stated, and so long as its vertical position is maintained when standing on end, its own weight cannot be added to the deflecting force, and hence it will need no more bracing than if suspended. It is *after* distortion has occurred that the effect of the weight of the cable or the arch is felt ; in one case to restore equilibrium, and in the other to increase its disturbance.

For railroad purposes, as well as to insure durability in the structure, this distortion, should be prevented as far as possible, and I have no doubt with a proper degree of stiffness in the roadway of both systems, the bracing of the upright arch can be quite as economically accomplished as that of the suspension cable. Just in proportion, however, as we permit greater undulations in the roadway of the bridge, the economy of bracing will incline in favor of the suspension system.

Recurring to Fig. 8: if half the span of the upright arch be loaded, a horizontal impulse is given to the arch at the crown, tending to move its central point towards the unloaded half. To prevent this horizontal movement of the centre of the arch, is one of the most important problems with which we are dealing. If the unloaded half be kept from curving upward, the movement of the center horizontally cannot occur, and then the undulation of the roadway will be reduced to the minimum. Evidently the slightest bending of the girder will result in a horizontal movement of the centre of the arch and cause a wave in the roadway, hence depth of girder is an important element in stiffening the arch. This movement naturally suggests as an expedient that the

ends of the lower chord of the girder be firmly fixed to the masonry, by which means one-half of the girder would resist this movement by tension, and the other half by compression, which would certainly reduce the wave. But owing to the effects of temperature, the ends of the girder, at the abutments, must be left free to move horizontally, or they will push or pull the masonry to its injury or destruction.

Another effect will be noticed as a result of the load being on half of the span; see Fig. 8. The top chord of the loaded girder, as well as the arch itself, is in compression whilst the lower chord acts in tension to resist the thrust from the upper chord; therefore, on this side we have two longitudinal compressive members, and as these strains are reversed in the unloaded half, we find two similar tension members there. It is evident that if the struts were run up to the upper chord and tension diagonals were introduced from the top of the first strut at the chord, to the bottom of the second one at the arch, and so on to the centre, the arch would supply the tensional resistance for the upper chord, and the lower one could be dispensed with in the loaded side and the tension chord in the other half. This modification simply substitutes spandrel bracing for the girder, and when jointed at the crown, forms the system used in the jointed arch bridge at Szegedin. From an examination of Fig. 10, which is a skeleton drawing of that bridge, the strains of the various members of one of its ribs will be seen, as given by M. Ritter, from whose work the drawing is copied.

The resistance to deflection by this and the preceding form of bracing would evidently be increased by greater depth of girder. In the Szegedin bridge the large sections of the braces and chords near the centre of the structure are due mainly to the shallow depth of the rib, while this depth has evidently been kept shallow, because of the increased length required for the struts and braces. These being alternately in compression as well as tension, are by economy limited in length. The span of the bridge is but 135 feet. In a structure of 400 or 500 feet length of span these members would be of great length, and hence the advantage of depth would be lost by its greater proportionate cost.

The half girder system of stiffening the arch will be found more economic than any method of spandrel bracing, because vertical members only are required between the arch and half girder, whilst greater proportionate depth is practicable without involving such long compression members. But for another important reason, greater economy and greater rigidity can be attained by it than by spandrel bracing. In resist-

ing the horizontal movement at the central joint by spandrel bracing, the diagram of the Szegedin bridge shows that the chord over the unloaded half arch acts wholly in tension. Its large sections in the central panels of the half rib, being about one-third of those of the arch, indicate the great pull brought by it upon the unloaded half arch, while stiffening it to resist the buckling impulse from the loaded one. This pull increases the compression in the unloaded arch, and thus increases the tendency in the central joint to move towards the unloaded side of the structure. When it is remembered that the deflection of the loaded half depends almost wholly on the horizontal movement of the central joint, it will be evident that any system of bracing which, while tending to stiffen the arch increases the impulse to move its centre in the direction of the unloaded half span, must be in conflict with economy, at least in theory, if not in practice. As the loaded half produces compressive strains at the same time in the chord over it, and these by the braces at the central joint also tend to increase its horizontal movement in the same direction, resistance to these strains can only be had by increased material. These objections do not exist in the girder system, whilst all spandrel braced arches are open to them, whether jointed or not. Therefore spandrel bracing will invariably prove more expensive, theoretically, than girder bracing, for the arch.

If the chords in the spandrel bracing referred to, be placed *beneath* instead of *over* the arch, as in Fig. 11, these objections vanish, because the strains are reversed. Under the loaded half they become tensile strains and resist the horizontal movement at the central joint by pulling directly against it, and this serves to prevent the spread or flattening of that half. By a system of struts, between this member and the arch above it, the chord becomes virtually a suspension cable and acts in unison with the arch to sustain the load, in the manner of the Saltash girder.

This brings us at once to the most economic solution of the problem, of preventing the horizontal movement of the centre of the arch, and as a sequence, to the most economic system of superstructure that is possible.

If we examine the effect of this arrangement on the unloaded side of the span, we find the chord here has become a compressive member and also resists the horizontal force at the crown. The intermediate strut work acts in tension and prevents the rising of the unloaded arch, as well as the fall of the chord. Exactly such strains in tension as are borne by

the loaded chord will be the sum of those borne by the unloaded one in compression. On the unloaded side, the arch and its chord or counter-arch act simply as a strut to transmit the horizontal and vertical forces at the central joint directly to the abutment at the unloaded side. The necessity of preventing, as far as possible by any method of bracing the arch, any horizontal movement at the crown, should not be lost sight of, as it is of the first importance in insuring rigidity. By no method can this be so economically accomplished, as by the counter or inverted arches shown in Figs. 14 and 15, which give also diagrams of strains of a 500 foot span on this plan.

By reference to these diagrams it will be seen that a notable uniformity of section is obtained in all of the members of the structure, a circumstance very favorable in construction. Perhaps the most remarkable result, however, that is developed by the diagrams will be the fact that with a steel arch, with maximum compressive strains of 20,000 pounds per square inch and all the other members of wrought iron, with 10,000 pounds maximum strains per square inch, and with 10 per cent. added for joints, it is practicable to sustain on a 500 feet span, a moving load of 2,500 pounds per linear foot by a superstructure weighing (including rails, floor bed and everything) only 1,500 pounds per foot.*

This remarkable result is due—

1. To the fact that every strain from temperature is completely eliminated.

2. By combining the arch and cable, great depth of girder or bracing is obtained.

3. No struts longer than one-half the versed sine of the arch are requred and but few that long.

4. No great strains, such as are incident in almost every other system, are thrown on the struts. When the roadway is suspended beneath the arch as in the case of the diagrams of strains (Figs. 14 and 15), the compression on the struts is much reduced.

5. Long struts being unnecessary, a greater versed sine or more economic depth of arch is practicable.

6. The least disturbance of equilibrium brings every member of the structure into play to resist it.

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* The correctness of this statement has been ascertained by carefully estimating the weights of the span from detail designs, and calculating the strains induced by the weight of the structure and its variously proportioned loads. The result given in Figs. 14 and 15, as well as the requisite sizes and weights of road-bed, rails, &c., have been all carefully verified by my friend Col. Henry Flad, and may be accepted with confidence.

I will not stop to compare the relative economy of this method of stiffening an arch, with that of any girder system as at present applied to suspension bridges, but will call attention to some of the facts disclosed by the diagram.

To span an opening 500 feet long with a bowstring girder, and attain equal stiffness would involve, as is shown by Mr. Barlow, the use of four times as much metal as the arch would require. The average strains in the arch in the diagram equal 1060.6 tons of 2,000 pounds, hence a bowstring girder would involve four times as much, and would require an average sectional area in its two members, equal to the resistance of 4,242.4 tons. To stiffen the arch in the diagram, an average section in the counter-arch is required equal to the resistance of 184.8 tons, or about $\frac{1}{23}$ part of the weight of the bowstring girder. Mr. Barlow obtained by his experiments with models a result of $\frac{1}{25}$ as the weight of his stiffening girder. The similarity of his experimental results, with those of the careful scientific deductions shown in the diagram, seem to me quite remarkable. It will be observed that his proposition as to the necessity of quadruplicating the weight of the girder to get the stiffness of the arch relates to the sustaining members only, and omits the bracing or web needed in all girders between the upper and lower chords. In his experiments this was of course included.

Respecting the relative strength required in a stiffening girder for a suspension bridge, as compared with the strength of a girder for the same span suited to bear an uniform load of the same intensity, Rankin says (in a note, page 375, "Applied Mechanics," 5th edition): "Hence, it appears that if the chain be supposed inextensible, the proportion borne by the strength of the stiffening girder to that of a simple girder of the same span, suited to bear an uniform load of the same intensity with the traveling load, ought to be as 0.138 to 1; while if the chain is supposed very extensible, as in the approximate solution, that it is found to be $\frac{1}{7}$ or 0.148 to 1; so that in the intermediate cases that occur in practice, no material error will be committed if that proportion be made as 1 to 7 or as 0.143 to 1."

The wide difference between the proportion of $\frac{1}{25}$ obtained experimentally by Mr. Barlow, and the $\frac{1}{7}$ mathematically deduced by Prof. Rankin, arises chiefly from the fact that the one relates to *stiffness*, while the other refers to *strength*. Mr. Barlow's illustration, shows that with double the metal and the same load, the deflection of the bowstring will be twice as great as that of the arch, but as the strains would not be increased thereby, it follows that double the metal in the girder should (theoretically) give the same strength of the arch, whilst four times the

metal is required to give the same stiffness. After making this allowance, however, the proportions of Prof. Rankin are nearly twice as great as those of Mr. Barlow. This may arise from difference in the assumed proportions of depth and length of girder and chain on which the Professor's calculations were based, and those of Mr. Barlow's models.

Leaving this discrepancy to be explained by others, I will point to the fact that by the diagram the average strains in the counter arches are only 184.8 tons of 2,000 pounds, while those in the arch are 1060.6, or $5\frac{3}{4}$ times as great. Hence, as it requires at least twice the metal of the arch to convert it into a bowstring girder of equal strength, it will be seen that the girder would weigh eleven and a half times as much as the counter arches by which the proposed arch is stiffened, or 65 per cent. less than the weight of stiffening girder required for the suspension bridge according to Rankin. While the stiffness of the arch over that of the bowstring girder is maintained under the whole load, the form and depth of the counter arches give far greater resistance to undulation under the movement of partial loads, than is possible by any practicable depth of stiffening girder yet proposed for a suspension bridge.

I am confident that a careful investigation of the system suggested and the facts stated, will convince those interested, that it is entirely practicable to brace the upright arch more effectually and with equal, if not greater economy, than is possible by any known method of stiffening suspension bridges; and that the proposed system avoids all the disadvantages resulting from temperature. These two difficulties have hitherto prevented the most perfect economy of superstructure from being attained. By overcoming them, the cost for long spans is wonderfully reduced, compared with the most economic truss systems yet devised. By any method of girder construction hitherto known, it is impossible to span a clear opening of 500 feet, with less than three times the dead weight of the arch on the proposed system, with equal strength of girder and with the same material and allowable strain. More than twice the quantity would be requisite in any case, but when the span becomes so great, a less economic depth of truss must be taken, and the length of truss must considerably exceed that of the arch, because the girder must rest upon the masonry, whilst the arch rests against it. In addition to this great excess in its cost, the girder will have twice the deflection of the arch under equal loads.

The arch, as hitherto constructed, being still much cheaper for the superstructure, it is evident that a great saving in the substructure must

have existed in the girder systems, to enable them to be introduced during the last thirty years in all parts of the world, almost to the exclusion of the arch. The difficulty of stiffening the arch, and the inconvenient effects of temperature, together with the greater cost of masonry, have given the different girder systems a degree of public favor which must disappear when these objections to the arch are removed.

Having shown how the cost of the arch in superstructure may be brought to the lowest possible point, by economic and effective bracing and by the avoidance of the effects of temperature, I will proceed to suggest such methods as will in almost every case, render in combination with the arch the economy of the girder *substructure* available. Evidently if the cheapest possible form of superstructure can be combined with the cheapest methods of substructure, we shall have attained the most economic system of construction that is possible.

The greater masonry required for the arch arises solely from the horizontal forces resulting from the weight of the arched span and its load, and from those which are induced by temperature. By reducing the weight of bracing to a minimum, and by eliminating the strains of temperature, we not only arrive at the greatest possible economy of superstructure, but by thus lessening the horizontal pressure upon the masonry to the lowest possible point, we also reduce the cost of the substructure to a minimum, so far as the arch *per se* is concerned. Therefore, to reduce still more the cost of masonry and approximate, if not equal, the economy of girder bridges in this item, we start from an already advanced point in our problem.

Where timber is abundant, an economic method of saving masonry in the piers of arched bridges may be employed where the bridge consists of two or more arches, by introducing wooden chords in them against the skewbacks or piers and abutments. These wooden chords would act in compression only and form a series of *compression* members instead of a line of *tension* members or chords from abutment to abutment, as in the case of bowstring girders. They need not, however, be in compression unless the bridge is loaded. If there be a series of long spans together, however, the loading of an arch at one end of the series would produce compression throughout the entire line of chords in the other arches, and this might shorten those chords so much in the aggregate as possibly to allow the loaded arch to spread too much, and thus produce objectionable deflection in the roadway of that arch. In this case it would be desirable to make the abutments stronger and put an

initial compressive strain in all of the chords of the system by means of screws or wedges against one end of each line of chord timbers.

In a series, for instance, of five arches of 500 feet span each, where the maximum horizontal force of the load is 500 tons on the chords, if an initial compressive strain of 400 tons be produced in the entire system, from abutment to abutment, when the bridge is unloaded, then this initial strain will be taken out of the chords of the first arch so soon as it has its maximum load on it, while the compression in those of the unloaded ones will be only increased 100 tons, and therefore the shortening of those chords would only be one-fifth part as great as if they had no initial compression; hence the deflection by load on any one arch in the system would be reduced accordingly, and would be really less than what it would be in an ordinary bowstring girder. In this case, the maximum stress on the abutments, when all the arches are loaded, would be 500 tons load + 400 tons initial compression + (the force from unloaded arches say) 300 tons, making a total of 1,200 tons; while the piers would be subjected to vertical pressure only, and hence they could be as cheap as if for ordinary girders.

If the timbers were secured together to resist tension, of course the compressive strains would be so much lessened, and the abutments proportionately reduced in cost. With such a system of wooden chords used only in compression, the repairs of the timber would be very simple and easy. The sticks should be squared at each end and butted against each other throughout the span, vertical movement being prevented by the connections of the floor beams with the arch, and lateral movement, by the wind bracing of the floor. To remove any defective stick it would only be necessary to withdraw the wedge, or slack the screw at the end of the line in which it was located, and by which the initial compression was created, and every piece in that line would then be released and any one easily removed.

The initial compression could be so great that no tension could be produced in either chord by wind pressure, and hence no jointing of the sticks together would be necessary to resist wind.

The track stringers and every longitudinal timber in the floor-way could be thus utilized to resist the thrust of the arches, and in this way, where timber is cheap, a very durable and economic structure can be erected. As no thrust in such a bridge can come on anything but the abutments, and as these can generally be located on the high banks of the stream, the cost of the entire substructure would exceed very slightly that which would be required in a truss bridge with spans of equal

length. It would probably be best to make the arches of such a series, uniform. The thrust at the abutments will be the same whether there be but one or many arches in the system. The stress on the chords (except initial) would be due entirely to the unequal loading of the various arches.

This method is applicable to parallel truss bridges, and by it the iron lower chord or tension member may be omitted and wooden compression members substituted therefor. The objection to the combination of wood and iron in bridge construction, owing to the difficulty of repairing the bridge, does not exist in this method. In all others, the wood is either under tension or compression and therefore difficult to be removed. In this, the entire chords of any one arch could be removed without endangering the stability of that arch or of any other one of the series; for it is plain, that if any temporary weight were placed on the floor beams, which would equal the weight of the chords to be removed, the equilibrium of the whole series would be undisturbed by their removal, so long as the whole bridge remained unloaded. In repairing it would never be necessary, however, to remove any one chord entire at once, but only to replace such pieces as were found defective.

Where the use of timber is found objectionable the cost of masonry in arched bridges may be greatly reduced by the following method also. Suppose *A B*, Fig. 2, be a bowstring girder, whose arch produces a horizontal force of 500 tons; and that the load increases this force to 1,000 tons. The iron chords of such a girder will require a sectional area of 200 inches if we allow a strain of 5 tons per square inch. Suppose two such girders, Fig. 11, constitute a bridge with abutments capable of resisting a horizontal force of 1,000 tons, and that each arch abuts against the other at the central pier, then it is evident that the section of the chords may be at once reduced one-half or to 100 square inches; for the thrust of one arch will balance that of the other, and the only strain which can come on the chords of either will be from the effect of the load, and this cannot exceed 500 tons. No horizontal strain can come on the central pier, for any unbalanced thrust will be borne by the chords. If both arches be equally loaded, or both be unloaded, no strain whatever will be on the chords of either.

If the arch *A* has its maximum load imposed, its chords will be strained in tension to 500 tons, but if the chords of both arches be formed to resist compression as well as tension, 250 tons of this strain will be transferred in compression to the chords of *B*, and by it to the

abutment of *B*, while the tension in the chord of the loaded arch will be reduced to 250 tons. Instead of a central pier which would have to sustain a horizontal force of 500 tons if no chords were used, we now have a pier subjected to vertical pressure only; and instead of bowstring girders, with chords of 200 square inch section, we need only one-fourth of that. As the chords of a bowstring girder are in great part supported by the arches, if we can reduce the chords 75 per cent., we at the same time lessen the requisite section of the arch itself, by relieving it of this much dead weight.

The only difficulty in carrying out this suggestion is the expansion and contraction of the chords by heat and cold. Suppose that to the end *A* of the lever *A B*, Fig. 12, we attach one end of the chord *C* of one arch, and to the end *B* we attach one end of the chord *D* of the other arch, and that the fulcrum *E* be secured to the central pier, whilst the other ends of the chords are attached to the abutment ends of the two arches. Now, if the inner ends of the arches be made to abut against the fulcrum of the lever, it is evident that the two chords may expand or contract without moving the central pier, or in any manner disturbing the arches. Such expansion or contraction of the chords will simply cause the lever to turn upon its fulcrum. A force against the fulcrum from either side of it will not, however, tend to turn the lever at all. Such force would be resisted equally by both chords, one in tension and the other in compression. One half of the force—500 tons due to the load on one arch—acting in the direction of the arrow, would be received on *C* in compression, and one-half on *D* in tension, whilst the only movement of the lever or fulcrum would be that due to the stress on *C* and the tension on *D*. The lever could be made in the form of two circular disks fitted in circular rests in the skew-backs, the axes of the disks being coincident. The end of the chord from one arch would be fastened between these disks by a suitable pin passed through that chord and the upper part of the disks, and the other chord would be fastened by a similar pin through it and the lower part of the disks. The effect of heat or cold on the chords would simply cause a rotation of the disks in their rests without strain and with no horizontal movement of them or the skew-backs. This lever arrangement is illustrated by Fig. 13, which shows a skew-back in perspective, seated on a series of rollers resting on the cap-plates of the pier. The object of the rollers is to avoid any possibility of horizontal strain on the pier, arising from the extension of one set of chords and

the compression of the others in two adjacent spans, when the load on one span is greater than that on the other. This movement will be so slight, however, in spans of ordinary length, that if the central pier were proportionately high, the rollers would be unnecessary.

The chords of the arches are, in Fig. 13, attached to the disks by links pivoted to them. One end of a link is inserted between the two disks, and a pin is passed through its end and through the two disks *above* the centre of the disks, while the link from the other chord is similarly secured by a pin passed through the disks and link *below* the centre of the disks. The other ends of the links are pivoted to the adjacent chords of the two arches. This, in effect, constitutes a vertical lever attachment for the ends of the chords, of greatly stronger design and more compact than would be possible with a vertical lever having a central fulcrum pin through it. The links must, of course, bear compression as well as tension.

This plan of disks is best used in two-span designs, but can be used with great economy in those of but one, or of more than two spans.

When both arches of a two-span bridge are loaded, the abutments will of course receive as much thrust as with an ordinary arch, for the chords are only strained when a greater load is on one arch than the other. But the central pier has nothing but vertical pressure on it, and may be greatly reduced in cost, while the chords are only one-quarter the weight that would be needed in ordinary bowstring girders. If four spans be desired, the central pier need then only be strong enough to resist the thrust due to unequal loads on the arches, as the thrust from the dead weight of the arch on one side of it will be balanced by that of the one on the other side of it.

In the case of a single arch, let us suppose that one end rests upon one abutment capable of holding but half the thrust of the arch, and at the other abutment we locate a skew-back, designed substantially as in Fig. 13. Let a chord be secured to the end of the arch, at the first abutment, and to the inner link at the other abutment. Now, if the end of the second link be secured in this abutment so as to resist compression, it will throw against this abutment one-half of the thrust of the arch, while the chord will receive the other half in tension. By such a modification one-half the weight of the chord of an ordinary bowstring girder and one-half of the masonry that an ordinary arch would require will suffice. The skew-back with the disks in it would have to move on rollers in this case, and would make one-half of such movement as would be due to the elongation of the chord from temper-

ature and from the load also. By very simple modifications of the system this movement could be divided so as to occur equally at both abutments. This movement would, however, be no more objectionable than that of the ordinary iron girders on their piers, while the deflection, by preventing the arch from spreading, would be 25 per cent. less with a single arch, and 50 per cent. less if two arches are used, than with any girder bridge of equal depth and strength now known.

By combining with this modification of the bowstring girder, the counter-arch method of bracing the arch, before suggested, and with central and abutment joints in the arch, we have a system of bridge construction from which the effects of temperature are absolutely eliminated, and which will be found to greatly surpass in economy of superstructure anything yet devised, and which admits of such reduced cost of substructure as to almost, if not quite, equal that applied to the various kinds of girder bridges.

The construction of the arches in half-length ribs, with the counter-arch bracing, enables the ribs to be easily erected, even if the spans be enormous in length. Each segment or half of a rib could be easily erected by a temporary tower placed in the stream, midway in the span, either on floats or piles, to support the inner ends of the segments, and on this, and on the abutments, should be placed machinery sufficient to lift a segment. When it is understood that such a segment, with its counter-arch and bracing for a span of 500 feet to carry 2,500 pounds per linear foot, weighs less than 50 tons, if the arch be of steel, and that the hoisting machinery needs only to lift each end of such a segment, or 25 tons, the ease with which such arches can be put together will be at once manifest.

No suspension bridge system, yet devised, possesses anything like the resistance to change of form which this does, owing to its great depth of bracing; while for equal length of span it possesses greater economy. The catenary to span the same opening, must be longer than the arch, by the diameter of one tower, as it extends from centre to centre of towers, while the arch will spring from the faces of the masonry. This advantage possessed by the arch, will quite compensate for the joints required in its construction, and which are not needed in wire cables.

Where great height of span is desirable, the cost of masonry to hold an ordinary arch may become so great as to forbid availing of its unquestioned economy. In such cases the chord with its compensating lever or disk attachment, at one or both ends of the arch, will relieve the piers of any desired amount of the thrust, just in proportion to the relative

lengths of the lever arms, or relative distances from the centre of the disks, at which the pins are located.

It is, however, when we compare this system with other parts of the suspension system, that its great economy over that system is seen. When we leave the great central span, the chief feature of nearly all suspension bridges, and examine the large sections that must be maintained in its cables over the reduced spans between the towers and anchorages, and compute the necessary weight and great cost of the masonry required to resist the tension of the cables, and compare these features with the shorter and more economic spans and light piers which this system admits of in the approaches, we perceive its remarkable economy over the best suspension system yet devised. To the economy and rigidity secured by the system proposed, we must add in its favor also the important elements of safety and durability, which are secured in a higher degree by using the material of the chief sustaining member of the structure in compression, instead of tension.

There is no limit except a financial one, to the length of span which may be safely constructed by this system, and spans of 1,500 or 2,000 feet will be found to be entirely within a practical or profitable limit of expenditure.

Figs. 16, 17, 18, 19 and 20 represent a bridge with two spans of 400 feet each, on the proposed system, for a double-track railroad. Table I contains a summary of quantities and weights of materials for its construction. Table II contains the quantities and weights of a bridge with five spans of 250 feet each, all the piers being of masonry. Figs. 21, 22, 23 and 24 illustrate the five-span bridge. Both tables have been carefully prepared from complete designs for both bridges. (See pages 235, 236.)

Mr. Charles Macdonald—I would not presume to discuss the paper just read without opportunity for full consideration, nor will I at this time attempt an examination of the argument presented in favor of the economy of the arch over the suspension system.

We, as engineers simply, too often loose sight in bridge construction of a very important element, the cost per pound of the material and the facility for erection, while instead we are considering—also an important element—the reduction of weight. Those who examine the St. Louis Bridge, must be impressed with the great accuracy in workmanship required to fully carry out the theory of the design. The most expensive material was used, to withstand the strains which was said in Capt. Eads' first report to exist in the members. All this increased the cost per pound of the structure, and I

doubt whether in the construction of another bridge, he will again adopt this design for the purpose of saving a few pounds of metal.

In my judgment, and from a practical point of view, great increase in lengths of spans generally seems unnecessary. In most cases the problem of covering a given space is best solved, and with greatest economy, when the cost of superstructure about equals that of its supports. I will refer to a recent instance; bids were asked for a bridge on the Cincinnati Southern Railway, over the Kentucky River, where the depth from the rails to the water was 275 feet, and the opening was 1,225 feet. Ten or twelve proposals were received, only one of which was for a suspension bridge, and that was the most expensive offered; the others were for spans varying in length up to 270 feet only. In this case the engineers who bid, did so with reference to cost, and without regard to "glory"; each in designing his structure endeavored to get the most economical arrangement of parts and length of span.

More attention should be given to the cost per pound, and a preference to shapes of iron most easily and cheaply made, and which are most economical in the end. There may be a little more or less difference in weight in consequence, but where the cost of the whole bridge is taken into account, it is hardly necessary to make so nice calculations as have been just read.

Mr. Ashbel Welch—I wish to endorse as strongly as possible what Mr. Macdonald has said about cost of structures, and to remind this body of Civil Engineers of a cardinal principle to be regarded always in practice—that engineering is best which most fully answers its purpose at the least cost.

Mr. J. Dutton Steele—I will mention a historical anecdote relative to the Suspension Bridge built by Telford over the Straits of Menai, which will be found recorded in the history of that structure. It was his intention to construct a cast-iron arch, such as he had put up in many places in Great Britain; to support the *cossions* during erection, he designed a suspension system, similar to that employed at St. Louis but more elaborate, which when complete, was found sufficient in itself without the arch, hence he abandoned his first design and put up a suspension bridge.

Mr. Willard S. Pope.*—In this paper is stated the broad and sweeping proposition, that "Upright arched bridges can be more economically constructed than is possible by any other method whatever, no matter

* This and the following discussions were presented at the meeting of the Society, held September 16th, 1874.

what length of span may be required;" the general reasons are given on which this opinion is based, and in explanation and corroboration thereof are appended diagrams and estimates of materials in upright arched bridges of various spans, in accordance with the designs.

This proposition is a startling one. During the past quarter of a century the necessities of commerce have stimulated bridge construction utterly beyond precedent, and consequently great and earnest attention has been paid to this specialty of engineering. All over the civilized world engineers have devoted their best capacities to the study of this subject, and the concurrent action of almost all has been in one direction. Without consultation with each other, and certainly without undue bias, reasoning out their conclusions from different data and by different processes, they have, with surprising unanimity, departed from the simple arch and gravitated toward other forms of structure. Of the multitudinous bridges for railroad purposes that have been constructed in metal during the past twenty years, those that are of the pure arch form are insignificant in number. The bridge at St. Louis is almost the sole modern example.

If the proposition so boldly affirmed is correct, it shows that the general tendency of modern bridge engineers is in the wrong direction; that they have been misled and are all astray. It is surely proper, therefore, to examine with care the reasons on which this revolutionizing statement is based.

Action and reaction are equal. In a structure of the kind known distinctively as the truss, the compressive action of one member is met by the tensile reaction of its corresponding mate, and so the superstructure is of itself in equilibrium. In the suspended or the upright arch, the direct reaction is taken by the substructure, which must be enlarged accordingly. Therefore, both substructure and superstructure must be considered together in making any comparison between the economic merits of different forms of construction.

Here let me commend the characteristic fairness with which the writer has met the case. He recognizes the fact just stated, and very properly considers the entire bridge as a unit. Whatever may thought of his conclusions and the reasoning on which they are based, he certainly confronts the problem boldly and squarely.

First, as to the difference between a suspended and an upright arch. Here, as the reaction in both cases is borne by the substructure, that element of cost may, for the purposes of the comparison, be disregarded.

In the suspended arch, the cables which carry the structure must be extended full size over and beyond the supporting towers to the anchorage behind them. Thus the actual length of the bridge is considerably greater than the effective length. This item of cost is saved in the upright arch, and so far, therefore, the comparison is in its favor. In many cases, however, short spans are required at each end of the main span, in which case the extended cables answer for such support, so that such extension is not necessarily a total loss.

But the cables or chains of the catenary are in tension, while the ribs of the upright arch are in compression. It is doubtless the fact, as stated in this paper, that iron or steel can be trusted with equal strains for equal sectional areas, whether in compression or tension, although the almost universal practice is to tax the metal less with the former than with the latter duty. But in compression the ratio between length and diameter is an essential element, while in tension this is not considered. To get the full value of the compressive capacity of the materials named, the length of the member must not exceed ten or twelve times its diameter. This at once introduces the necessity of bracing the upright arch to secure its permanence, a necessity which does not at all apply to the suspended arch.

That we may get a clear insight into this matter, let us leave entirely out of view for the present the lateral strains induced by wind or by moving loads, inasmuch as these are common to both forms of structure, and material must be provided to resist them alike in each. Let us also disregard for the present the necessity (also common to each) of providing for a variable load, and assume that the weight to be carried is definite and uniformly distributed. The data being the same, the strains are alike in each, only of an opposite character; and it may be granted for the sake of the argument that the net sectional area of metal and consequently the net weights are also alike.

What, then, are the conditions respectively? Each single cable of the catenary is complete in itself. To perform its functions it needs no other or auxiliary connections. It is entirely independent, self-supporting, and carries its load without outside help. It is of simple construction, compact, solid, without objectionable joints, and contains within itself no element of destruction. It is easily and safely erected in place. Application of reasonable load, and indeed its own weight, serves only to increase its stability. All the forces of nature act with it, and it is in entire harmony with the immutable laws that govern all matter.

In the upright arch, most of these conditions are reversed. Each rib is by itself utterly helpless, even to carry its own weight, and must, therefore, be connected with other ribs by an elaborate and expensive system of lateral bracing. Its construction must be tubular, or rectangular, or of some similar form suitable to resist flexure, and all such forms are expensive. It abounds in joints, each one of which must be carefully and perfectly fitted. The lateral bracing must be so complete as to thoroughly secure the various ribs at intervals of not greater than ten or twelve diameters. To erect it in place is a difficult and dangerous task. The centre of gravity is high relatively to its base, and, consequently, the forces of nature act against rather than with it. No human work can be perfect, and defects of workmanship, that would be almost immaterial in the catenary, become serious when in the upright arch. It is impossible, even if it were desirable, to make a structure which will be absolutely rigid; oscillations and undulations that would be harmless in the catenary might prove ultimately fatal to its rival.

These are some of the inherent and broad differences between the two forms of construction. Other matters, such as provision for variable load, for side strains from wind, &c., &c., are common to both, and so need not enter into the present computation. And, in the opinion of modern engineers, it is these differences that have so generally influenced the substitution of other forms in place of the arch.

Indeed, the fact is not to be disguised that, on general principles, most engineers prefer to avoid compressive strains in metals as much as possible. It is easy to know perfectly our material and all its capacities in tension; but, in the present state of our knowledge so many elements of uncertainty enter into the use of iron in compression, that such use must be attended with much risk. It is one of those subjects concerning which we feel that a little knowledge is a dangerous thing, and prudent engineers are careful to err designedly on the side of safety.*

If an upright arch is to be used at all, the suggestion of a counter-arch for variable loads has much merit. The difficulties that were encountered in the designs of the St. Louis arch-bridge, from the effects of changes of temperature, are fully and forcibly detailed. In this

* The great desirability of more information in regard to the practical compressive capacity of metals is illustrated by the action of the Society in appointing a committee (of which Capt. Eads himself is a member) to procure the assistance and co-operation of the Government in making a thorough and elaborate series of tests and experiments on the capacities of metals. No more important subject than this can engage the attention of the Society and its members.

structure, the two members composing one rib are intended to act together, each bearing its proportion of compression, and at medium temperatures they do so act, but in extreme cold weather the rib deflects, changing of course its curvature, and then at the crown the upper member and at the springing the lower member, does all the duty, which duty is still further increased by an actual tensile strain brought upon the opposite member at those points. In extreme hot weather this action is reversed. Thus the novel experience is had, of certain portions of an arch subjected to tensile duty. This tension was found so considerable as to render it necessary to firmly bolt the skew-backs to the masonry. Doubtless the counter-arch jointed at the crown and at the skew-back would have obviated the difficulty, and indeed, it would seem that, even with the present construction, if the two members composing the rib had been brought together at the springing and at the crown, the trouble would not have occurred. As the bridge is designed, it appears impossible to tell how much duty each skew-back is at any time actually performing; just as it is impossible to tell precisely how much weight comes upon each leg of a four-legged table.

Accompanying Capt. Eads' paper are plans and estimates for a bridge on his design, with a span of 400 feet, and a rise of 50 feet, the floor being suspended below the arches flush with the springing. A thorough system of lateral and diagonal bracing between the respective arches and counter-arches is shown, so thorough indeed, as to quite block up the roadway, the lateral strut between the counter-arches nearest the springing being only about 6 feet above the floor, while the lateral ties seem to extend quite down to the floor.*

But assuming that a clear headway of 20 feet above the floor is needed for the passage of trains, no lateral strut or tie could be placed between the main arches for a distance of 40 feet from the springing, or between the counter-arches for a distance of 120 feet from the springing. For these distances therefore, the arch and counter-arch could only be stiffened laterally by knee braces from the floor; inasmuch as the entire floor is simply suspended from the arches this would not prove a very reliable security, and for just this locality where the lateral bracing is the weakest, the need of it is the greatest, as it is exactly here that the strains are the heaviest.

Indeed, the demands of an upright arch for continuous and effective lateral bracing between the ribs are so imperative and absolute that it

* This, however, is evidently an oversight of the draftsman.

is almost a necessity that the flooring and passage way should be placed either entirely below the springing or above the crown. The vertical space between the springing and crown is fairly interdicted for such service, and any intrusion therein is at the expense of the stability or economy of the structure. The grade line of the floor being generally the governing consideration, if the arches are placed above it sufficiently for the necessary head-room, they are raised very high, which of course adds to masonry, and so to cost. If the arches are placed below the grade line, they not only take up what may be valuable headway for navigation but also necessitate a heavy and costly system of struts for the support of the roadway on top of the haunches.

It is furthermore pertinent to consider that so far as bridges are or may be built over navigable streams, an arch placed below grade usurps more of the head-room for vessels than does any other form of structure. A law decreeing a certain specified clear space between high water and the crown of the arch may prove very deceptive as a protection to the interests of navigation.

Leaving, now, the comparison between the upright and the suspended arch, we come to a consideration of the devices suggested to secure economy in masonry. These are ingenious, and so far as I know, novel; they can be best understood by a reference to the paper. That the line of timber chords suggested for a series of arches would answer the purpose I have no doubt; to secure them firmly however, would require a large amount of bracing, both vertical and lateral. They would not be especially ornamental, and would be liable to decay and to fire. To decide as to their ultimate economy would require a calculation for each particular case. I imagine that engineers generally will hardly urge their adoption.

The other and more elaborate plan provides for dividing the thrust into parcels, and apportioning a certain definite part to the abutments and a certain other part to a system of longitudinal tie-rods or lower chords; in other words, the arch now becomes partly an arch, and partly a bow-string girder. In the case of two adjoining spans, if they are both empty or both uniformly loaded, the abutments receive the entire thrust and the chords carry nothing. But if one arch is loaded and the other empty, then the chords are strained to the extent of one-half of the thrust from the live load, and the arrangement is such that of this thrust one half is borne in tension by the chords of the loaded arch and the other half in compression by the chords of the unloaded arch; the chords

being so constructed as to resist both tension and compression. In any case the pier sustains nothing but vertical load. The device is ingenious in the extreme and well worth careful study.

The reflection presents itself however, that possibly there may be some practical difficulty in harmonizing three distinct agents, viz., the abutments in gravity, the lower chords of one arch in compression and of the other in tension, so that each will always do exactly its allotted duty, for if one does less, the other must do more, and thus be overburdened. It is somewhat similar in effect to a continuous bridge resting upon several supports, so long as each does its work fully and fairly, so long is the structure effective; but if one settles or in any way shirks its duty, new, unexpected and dangerous strains are at once developed in the girder. Another pertinent illustration of the difficulties attending a divided duty, is found in the practice formerly so common of uniting an arch and a truss in the same bridge. Experience showed that it was impossible to harmonize them—one did all the work, and the other, instead of being a help, was really an incubus and a nuisance; the better modern practice has, I believe, entirely abandoned the plan.

Even assuming that it is entirely practicable to parcel out the thrust of the arch, as it is proposed to do, yet the plan will not be without cost; inasmuch as a lower chord 400 feet long, to resist even limited compression, must be well and thoroughly braced both vertically and laterally. If it were simply suspended from the arch as shown in the drawing, it would be as limber as a whip-lash.

After all, it is largely true that in such matters each particular locality forms a law unto itself. A structure that is admirably adapted for one place may be utterly unsuitable for another, and he is the best engineer who best fits his plans and designs to the peculiar demands which they are to serve, and who thoroughly accomplishes the desired purposes with the least expenditure of money.

The entire paper is exceedingly interesting and suggestive. It is a bold and vigorous attack upon the labors and opinions of the great majority of modern bridge engineers, and both for the brilliant professional repute of its author, and for its own merits, it will command and receive earnest attention.

Mr. Thomas C. Clarke—It must be a source of gratification to the Society that a member, so much occupied with public business as Capt. Eads, should find time to present such an elaborate paper as this.

During the last seven or eight years the proper construction of iron

bridges has been investigated with great ability. It will be observed, however, that almost the whole of this effort has been directed towards one point—elucidating the action of the strains caused by a load upon the main trusses or girders. Very little attention has been given to the bridge as a whole—that is regarding it as a complex piece of framing, subject to impact, vibration and the swaying from side to side, caused by a rapidly moving train ; unless all these points are carefully attended to, the bridge cannot be considered a perfect structure—time will develop its defects.

It is with some hesitation that a criticism of the plan here presented is ventured upon, from this point of view. Assuming for the present, that the main arches do what is here claimed for them—require less material than any other system—yet it will be seen that a platform merely suspended from them will not possess much lateral stability, and must sway from side to side under a rapidly passing load. In what is termed a "through" bridge, the floor is held in place not only by its iron system of horizontal bracing, but also by the posts and knee or angle braces which connect it with the upper chord. Horizontal bracing alone will not prevent lateral motion, for the lower chord being in tension will vibrate like a stretched string. It is necessary to connect the floor by rigid angle bracing with the upper chord, which being in compression, never leaves a straight line. If this were done in the bridge proposed by the writer, some of the economy of material claimed for it would disappear.

Moreover, practice has shown that a bowstring girder is the worst possible form to brace against horizontal motion, on account of the very acute angle at which the ends of the arches meet the chords. Practice has also shown that the deflections on a bowstring girder are very great at each end, owing to the want of height of the truss at these points. The locomotive seems to fall upon it with a blow or shock on leaving the pier, instead of gliding on over a curve gradually increasing toward the centre, as in a parallel girder.

In its present shape, the bridge proposed is not well suited for a rapidly moving load ; it may possibly be modified, but this will add material again.

In the next place, the economic merit of this system does not appear to be borne out by the figures presented. The weight of five double track spans of 250 feet each, in iron, is estimated at 3,010,300 pounds. The weight of five equally strong parallel girder spans would not exceed

3,300,000 pounds, and on account of their greater simplicity of construction, would cost enough less per ton to more than compensate for the difference in weight. If we add to the plan as it now is, enough iron to make it laterally stiff, the cost would somewhat exceed that of the parallel girder.

But when we come to consider the quantity of masonry required by the two systems, we see the great economy of the parallel girder. The masonry in these two piers (which, like those under a parallel girder, receive no pressure except a vertical one) is estimated at 1,410 cubic yards, while those two which receive the semi-thrust of the arches require 3,630 cubic yards, or more than double as much. It is difficult to see wherein this lever-disk arrangement, which by the paper under consideration seems to save half the iron in the chords and double the amount of masonry, is so economical.

Finally, the conclusion to which we arrive is this—economy in bridge construction is to be attained by saving in the cost of piers and abutments so as to allow of shorter spans, rather than in the cost of iron in order to build very long spans. There are of course, other questions than those of economy which come into consideration, such as exceptionally long spans required in navigation, or because good pier foundations cannot be obtained, but this is foreign to the subject of the paper, which challenges the parallel girder system on the ground of *economy alone*. We say that the case has not been proven; because the cost of the piers and abutments which should make up at least one-half of the cost of the whole—but which in the case of this system, would absorb considerably more than one-half—has been omitted.

Mr. Louis Nickerson—In discussing the bridge before us, it would be well to examine somewhat the short review of arch bridges by Mr. Barlow, which has been used as a postulate. He therein claims, that for certain reasons, an arch bridge is more economical than a truss of triangular or trapezoidal elements. I think that this is an error. For even as a theory, it is founded on the assumption that the deflection of a truss of any kind is the manifest point, from which the investigation should be made. Now the Fink bridge, though rather more costly in original construction, than some other well-known forms, is one of the cheapest as regards maintenance, and one of the safest as regards stability, that is used. Yet the Fink bridge deflects more than any other known bridge; I think twice as much as the Warren girder improved by Fink. But the same cause which makes its entire deflection great—the

advance of the stress towards the center, and the uniform flexure of the chord from compression—entirely eliminates the reverse curve common to other bridges, which it is most our duty as engineers to get rid of, and which, so far as my knowledge extends, is much worse upon arch bridges, metal for metal, than upon any other. It is this partial and wave-like deflection preceding and following the engine in its course, which breaks the back of the bridge, destroys its fastenings and abutting surfaces, and which engineers have attacked since the substitution of more elastic material for stone. The design before us really strives against this, with an effect which we will attempt to measure hereafter.

As regards practical economy, the case seems to be as follows: place a bowstring bridge on piers and a similar simple arch between abutments (their stiffening webs may be supposed equal), then if the chord of the bowstring costs more than the extra abutting masonry, the open arch is the cheapest; if the chord cost less, then the open arch is the dearest. This refers to the amount of materials alone.

In constructing an ordinary triangular or trapezoidal truss bridge, it is not necessary that the handicraft be very perfect. An error of $\frac{1}{64}$-inch in length is not perceptible in practice, and as regards safety more may be allowed with impunity. By bringing to bearing little imperfections it will rather improve than otherwise during a year's service, after which they seem not to exist. Now, to arrive at this $\frac{1}{64}$-inch costs but little per pound of material, because it is easily approximated by ordinary machine tools, but to finish with the same perfectness the abutting surfaces in the long and by itself flexible curved member and also the bracing members of an arch bridge, requires much more care, and even when not carried to scrupulous excess, involves a much greater working cost per pound; whence a rectangular bridge can be built at less first cost, and all things considered, with a really more economical result than can an arch bridge.

If you were asked "what is the lowest priced bridge, so far as amount of material is concerned, that can be built?" I think the unanimous answer would be "that rectangular bridge which, with the greatest height of truss is least liable to 'buckle' the same area of web"—the Whipple bridge, and we know (whatever faults it may have) the Whipple bridge is not only cheaper in cost, but, metal for metal, has much less haunch deflection than an arch bridge. Arch bridges, as hitherto constructed, can neither be considered economical nor very rigid.

Notwithstanding his use of Mr. Barlow's text, the author of the

bridge before us realizes the difficulty of the reversed curve in the chord; this is shown by the fact that he has made no special attempt against the bridge deflection, but has struck strongly and well against the haunch deflection, and whether successful or not, he deserves our praise for the essay. Fortunately for your patience, the best way to arrive at the measure of this is the most simple and expeditious one.

First, in regard to stiffness, let us remember a fact too often forgotten—that no system of bracing, however complicated, can have any other influence upon the effects and directions of the forces in a girder, than the most simple plan that an engineer could imagine. If the braces be removed from the "loup" girders of this bridge and replaced with plain plates of the same resistance, the status of the girders or the bridge has not been in the least altered. Take an ordinary bowstring bridge of the same span and height—a similar bridge—and replace the web of that with a corresponding plate; we could mark upon it with chalk the exact saving of metal with but little computation,—and from this deduce the form of the two "haunch" girders.

Remember, that the rigidity or flexibility of a bridge does not at all depend upon its height of truss, *but on the depth of web within the truss*, and as no form of bracing within the limits of a truss, however ingeniously conceived, can vie in depth with the web that fills the truss, therefore none can be so stiff. With arch and bow-string girders then—which are already in fault with respect to flexibility—the change is not an improvement.

Second, in regard to strength;—this quality and that of stiffness are fully separated in the design. As we have only been called on to treat of one in discussing the arch arrangement, so we are almost exclusively confined to the other when we come to examine the lower chords and lever attachment. That the arrangement is highly ingenious all must at once see, yet it seems to me it must fail from the same cause that several attempts of the same kind have failed—notably, the construction of a trussed girder of which the beam should bear one-half the load and the rods the other portion, or with the same idea the attempted combination of arch and truss.

For, supposing first only tensional chords, and that the loaded span would elongate and its arch move outwards against the heel of the other, as described in the paper, the unloaded arch would then move slightly inwards, thereby being relatively weaker to resist thrust than before, and an additional strain would thus be thrown upon the chord of the

loaded truss. Then this must further elongate, the loaded arch spread, and the unloaded arch be again forced inwards, becoming still less capable to resist thrust; so repeating until *the security being sufficient*, the engine would cross the first arch, strike the second and cause it to recoil with a jerk; the same action to be partially reversed upon the other when the load had entirely passed onwards. Safety then would probably depend upon there being sufficient *security* to allow the chords to resist the *full* horizontal strain.

But supposing the chords capable for compression (and they would then probably cost as much as full-sized tensional chords), we must remember the inequality between the elongation and depression of material within elastic limits, and even at low strains, so that to shorten one chord a certain distance, and to lengthen the other the same distance (a *sine qua non* to the attempt), would require very different amounts of force (per square inch), a fact at once seen to be inadmissible with equal chords. Hence, at no moment could there be a mutual assistance.

In the two cases mentioned, the trussed girder and the arch and truss, the first was too obviously in error to hold a place in mechanics long, and in regard to the second, it has been found necessary and more economical, to make one or the other member sustain the load, because by no combination would the materials act together in such unison as was required. It seems to me, with all diffidence, that in the case before us, either a sufficient chord or a sufficient abutment is necessary for safety and economy.

Mr. Samuel H. Shreve—The paper before the Society appears to be an acknowledgment of "the disadvantages of the system adopted for the superstructure of the St. Louis Bridge," which the writer says were forcibly thrust upon his attention in the progress of that work.

He says of the bridge: "If the effects of temperature could be avoided, and the curve were a parabola, an equally distributed load on a rib would, of course, be borne by the upper and lower tubes equally, that is half on each"—but the effects of the temperature are inevitable, and consequently the above supposition, if correct (which it is not), is without value. He further says: "This extension (from the temperature) causes the crown to rise, which relieves the lower tube of compression at the abutments, and hence that tube does not then support any portion of the weight of the rib or the load. * * At the crown, the strains are likewise changed. There, however, the *lower* tube does all the duty, as the upward bending of the rib relaxes the compression in the upper tube at this point."

Suppose the crown to rise, the lower tube if relieved of compression at the abutments, would not be strained at any point. Bracing beneath a parabolic arch unformly loaded can receive no strain and is useless, and if the curve of the tube considered is not a parabola, it is so near it, that the strains which are transferred by the braces to the lower tube are too small to be noticed. The conclusion is unavoidable, that the upper tube at times must bear the whole load, and even more, for the tubes "are rigidly held to the abutments by anchor bolts," and a large extra amount of strain is in this manner brought upon the loaded tube. Now consider the opposite condition: "As the ribs are formed for a medium temperature, contraction by cold reverses these strains—the crown of the arch falls," and the lower tube must just as surely take the whole load and the upper one be useless. Suppose a still different case; when the tubes are affected by the temperature, one may, and probably does, expand or contract more than the other, and the condition is still worse.

The result is simply this: each tube must contain a quantity of material sufficient to resist the strains caused by a full load, and more (on account of the bolting down of the abutments) or be injuriously strained.

This doubt as to which members the strains will affect should not exist in regard to any truss, and the axiom may be laid down, that a plan in which such ambiguity of strains obtains is sufficiently defective to be rejected without further examination.* This complication may be prevented in a simple manner, and the strains confined to their respective members which then need not be proportioned to bear all that can possibly come upon any part of the truss.

Suppose that of a framed arch the lower arc member at each abutment, and the one of the centre are removed, the structure will then be of the form shown in the figure (next page), divided into two parts, jointed together at the centre and similarly joined by the same tube or arc to the abutments. Expansion and contraction will not affect this truss, and the upper arc has its work well defined, which work cannot be added to by any action on the part of the lower arc.

The horizontal component of the strain in the upper arc is

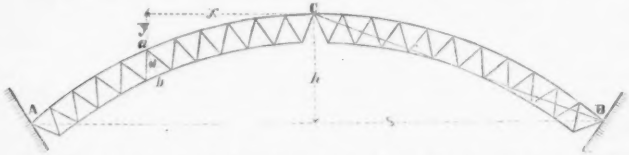
$$H = \frac{w\,s}{8\,h}. \qquad (1)$$

where w is the maximum load uniformly distributed, s the length of the

* There is no difficulty in calculating the strains in a correctly planned arch truss; they are readily determined by simple mathematical reasoning as in any horizontal truss. The labored and uncertain theory of arches as applied to masonry, has no more bearing upon arched trusses than it has upon a Howe truss or a king-post bridge.

span and h the height of the centre of the arch above the points where it is joined to the abutments.

The segments of the lower arc are sometimes subject to compression and sometimes to tension, according to the position of the load. The braces are needed only under partial loads. That the greatest tension in the lower arc of either segment is when that segment alone is fully loaded, is self-evident; to determine the amount of strain and the requisite quantity of material, some calculation is necessary.



Let the arch be loaded uniformly (horizontally) from A to C, then since w represents the full load of equal density, the weight borne by the abutment B, or its vertical reaction, is $\frac{w}{8}$. The load upon $A\,C$ produces a strain which must pass through C and B, and which may consequently be represented by the strain line $B\,C$, of which force h will represent the vertical, and $\frac{s}{2}$ the horizontal component. This vertical component is $\frac{w}{8} : h :: \frac{s}{2} :: \frac{w}{8} : \frac{w\,s}{16\,h}$;

hence $\frac{w\,s}{16\,h}$ is the horizontal component of the force through $C\,B$, or the horizontal reaction of the abutment B: it also is evident that there are the same forces acting at C as at B.

Take moments at any point a, where the braces meet in the upper arc in the loaded part, distant x from the centre C, and we obtain for the longitudinal strain in the opposite member of the lower arc, $L \times d$, (d being the depth of the arc) = the vertical reaction at $C \times x$, which = $\frac{w\,x}{8} + \frac{w\,s}{16\,h} \times y$ (the term being the horizontal reaction at the same place, $\times y$ the vertical distance of C above the point a) $- \frac{w\,x^2}{2\,s}$ (the load on x); collecting these we have

$$L\,d = \frac{w\,x}{8} + \frac{w\,s\,y}{16\,h} - \frac{w\,x^2}{2\,s}. \qquad (2)$$

Since the curve is a parabola, y may be expressed in terms of x, thus: from the properties of the parabola, $x^2 = 2\,p\,y$, and $\frac{s^2}{4} = 2\,p\,h$

$\therefore\ 2\,p = \frac{s^2}{4\,h}\ \therefore\ y = \frac{4\,h\,x^2}{s^2}$, hence

$$L\,d = \frac{w\,x}{8} + \frac{w\,x^2}{4\,s} - \frac{w\,x^2}{2\,s},\ \therefore\ L = \frac{w\,x}{8\,d} - \frac{w\,x^2}{4\,d\,s}. \qquad (3)$$

From this equation we can find the strain in each member of the lower arc of one segment when that alone is loaded. It is seen that when $x = \frac{s}{4}$, L reaches its maximum value and becomes $= \frac{w\,s}{64\,d}$.

For convenience in calculating, let d bear a certain proportion to h, say $d = \frac{h}{4}$, as this is somewhat near the facts in the St. Louis Bridge, and then

$$L = \frac{w\,s}{16\,h}. \qquad (4)$$

As the load passes on and gradually covers the whole arch, this tension diminishes until when the arch is fully loaded, it entirely disappears. When the arch is half loaded, as in the case supposed, the upper arc in the loaded part is subject to much less compression than when it is fully loaded. When one segment is loaded, the upper arc of the other is subject to tension and the lower to compression; a glance at the figure will show that the pressure from C in the direction of B will tend to cause a bending upward of the segment $C\,B$. Here again we can easily find the strain; take the moments around a point distant x to the right of C, the left segment being loaded, and we have the same equation as before,

$$L\,d = \frac{w\,x}{8} - \frac{w\,s\,y}{16\,h},$$

the forces at C acting in opposite directions.

As the load comes upon this segment, these strains gradually disappear, and unlike the previous case, are diminished by the weight of the arch. The tension in the upper arc can be entirely prevented by giving sufficient depth to the arch. To do this in the St. Louis Bridge, 12 feet will be found to be ample; that is, a depth of 12 feet in this arch will prevent any tendency in the unloaded segment to rise when the other segment is loaded.

These investigations have been made to obtain formulæ whereby the quantities of material may be compared. The cubical quantity of material in any member of a truss is the area of its cross section multiplied by its length, and the area of the cross section varies directly as the strain upon it, so that if the strain is multiplied by the length of the member supporting it, the product will be all that is needed for the purposes of comparison, and a quantity from which the weight may

be directly calculated. For compression members a certain percentage to prevent buckling or bending must be added, which generally may be put at, say 25 per cent.; in tension members this addition is unnecessary, and in comparison no extra material need be allowed for joints, as this may be assumed to be the same in the different cases.

In the centre arch of the St. Louis Bridge, the span is 515 feet, the rise at the centre 51.5 feet, and the length of the arch in even numbers is 529 feet, or about 2½ per cent. longer than the span. The horizontal component of the strain throughout one arc or tube when fully loaded is $\frac{w\,s}{8\,h}$, and bears about the same proportion to the longitudinal strain that the span does to the length of the tube, or the longitudinal strain,

$$L = \frac{1.025\ w\,s}{8\,h}, \tag{5}$$

multiply by the length of the tube, 1.025 s, and we have the quantity,

$$Q = \frac{1.0625\ w\,s^2}{8\,h}; \tag{6}$$

next add 25 per cent. for stiffening, and

$$Q = \frac{1.0625\ w\,s^2}{8\,h} + \frac{1.0625\ w\,s^2}{32\,h} = \frac{5.3125\ w\,s^2}{32\,h}. \tag{7}$$

Now, as each tube of the St. Louis Bridge is liable to be subjected to the whole load, the quantity of material required for the two tubes in one rib, is

$$Q = \frac{5.3125\ w\,s^2}{16\,h}, \tag{8}$$

Compare this with the quantity of material in an ordinary horizontal truss of equal length and depth. The quantity in either chord is represented by the contents of a parabola, whose base equals the length of the truss, and whose height is equal to the strain at the centre of the truss, or,

$$Q = \frac{w\,s}{8\,h} \times \frac{2\,s}{3} = \frac{w\,s^2}{12\,h};$$

add the 25 per cent. for the upper chord, and

$$Q = \frac{5\ w\,s^2}{48\,h}; \tag{9}$$

for the lower chord,

$$Q = \frac{w\,s^2}{12\,h}. \tag{10}$$

Adding (9) and (10), and

$$Q = \frac{9\ w\,s^2}{48\,h}. \tag{11}$$

Comparing these quantities we find that the chords of the horizontal truss require less than 57 per cent. of the material needed in the ribs of the arch. I do not know how much material the tubes contain,

but if they are properly proportioned to bear the loads, there must be that difference, which will hold good if the arch and truss are subject to the same strain per square inch. This enormous difference being considered, it is hardly worth while to discuss the comparison of the quantities in the braces. No argument further is needed as to the merits of these two systems.

To compare the arch of the St. Louis Bridge with that of the figure, we have, in the upper arc of the latter, the same quantity as in one of the tubes of the former, or

$$Q = \frac{5.3125\ w\ s^2}{32\ h}.$$

In each segment of the lower arc the strains vary as in a horizontal truss, and are equal to the contents of a parabola whose base $= \frac{s}{2}$ and whose height $= \frac{w\ s}{16\ h}$, hence $Q = \frac{w\ s}{16\ h} \cdot \frac{2}{3}$ of $\frac{s}{2} = \frac{w\ s^2}{48\ h}$,

and for two segments, $$Q = \frac{w\ s^2}{24\ h}; \qquad (12)$$

add this to the quantity for the upper arc, and

$$Q = \frac{19.9375\ w\ s^2}{96\ h}, \qquad (13)$$

which is 62 per cent. of the material in the St. Louis arch. The horizontal truss has still the advantage even of the arch properly constructed; the latter does not need so much material in the braces, but the roadway is to be supported, and other expensive items added that will probably more than compensate for the difference.

The form of arch which the writer now recommends is certainly a great advance upon his previous one, but it is doubtful if it possesses great advantages over an ordinary bowstring. His views in this particular seem rather extreme, and will hardly be accepted by engineers, who are likely to be satisfied, for the present generation at least, with what has already been done in the construction of steel arches.

TABLE I.

Estimate of Weights and Quantities of Double Track Railroad Bridge of Two Spans, each 400 feet between Skew-back Pins; with Lever Arrangement.

Weight of structure for a main arch of steel—1.28 tons per foot, and of wrought-iron—1.93 tons per foot. Live load—2.5 tons per foot. Versed sine—$\frac{1}{8}$.

1st. Main arch of steel. (Ten per cent. having been included for weight of joints.)

DESCRIPTION.	Steel.	Wrought iron.	Cast iron.	Timber.
	Pounds.	Pounds.	Pounds.	Pounds.
Weights in one span—Main arch	253,630	.....	.....	.....
Counter arch	.....	110,880	.....	.....
Main bracing	.....	147,550	.....	.....
Lower chord and track stringers—				
Chord in centre	.....	48,720	.....	.....
2 outside chords	.....	84,800	.....	.....
4 track stringers	.....	137,600	.....	.....
Cross beams	.....	60,523	.....	.....
Wind bracing of main arch—				
Struts	.....	20,000	.....	.....
Rods	.....	46,466	.....	.....
Diagonals	.....	10,000	.....	.....
Wind bracing of tracks	.....	27,048	.....	.....
Suspension rods	.....	21,000	.....	.....
Cross-ties and planks	.....	.....	.....	25,440
Rails	.....	32,000	.....	.....
Joint in centre	.....	.....	7,000	.....
Total	253,630	746,587	7,000	25,440
Total weight of span 1,032,657 lbs. = 1.29 tons per lin. foot.				
Weights of both spans	507,260	1,493,174	14,000	50,880
Castings above piers	.....	.....	63,000	.....
" " abutments	.....	.....	36,000	.....
Ends of main arches	.....	.....	20,400	.....
Levers and pins	4,620	.....	.....	.....
3 columns of centre pier	.....	.....	141,000	.....
Wind bracing	.....	20,000	.....	.....
Lanterns	.....	.....	4,500	.....
Total	511,880	1,513,174	278,900	50,880
Masonry (estimated from 50 feet below superstructure)—2 abutments, 6,506 cubic yards.				
2d Main arch of wrought iron:				
Weights in one span	.....	1,520,587	7,000	25,440
Total weight of one span 1,553,027 lbs. = 1.94 tons per foot.				
Weights of both spans	.....	3,041,174	14,000	50,880
Castings on pins and abutments	.....	.....	99,000	.....
Ends of main arches	.....	.....	20,400	.....
Levers and pins	4,620	.....	.....	.....
3 columns of centre pier	.....	.....	141,000	.....
Wind bracing	.....	20,000	.....	.....
Lanterns	.....	.....	4,500	.....
Total	4,620	3,061,174	278,900	50,880
Masonry (estimated from 50 feet below superstructure), 2 abutments, 7,386 cubic yards.				

TABLE II.

Estimate of Weights and Quantities of Double Track Railroad Bridge of Five Spans, each 250 feet, between Skew-back Pins; with Lever Arrangement.

Weight of structure of main arch of steel—1 ton per foot; and of wrought iron—1.2 tons per foot. Live load—2.5 tons per foot. Versed sine—.

1st. One Side Span. (Ten per cent. being allowed for joints.)

Description.	Main Arch of Wrought Iron.			Main Arch of Steel.			
	Wrought iron.	Cast iron.	Timber.	Steel.	Wrought iron.	Cast iron.	Timber.
	Pounds.	P'nds.	P'nds.	P'nds.	Pounds.	P'nds.	P'nds.
Main arch	183,000	.....	.....	86,000	.....		
Counter arch	44,500	.....	.....	.....	44,540		
Main braces. Struts	24,000	.....	.....	.....	24,000		
Rods	29,880	.....	.....	.....	29,880		
Joints	5,000	.....	.....	.....	5,000		
Lower chords and stringers	169,250	.....	.....	.....	169,250		
Cross beams	41,700	.....	.....	.....	41,700		
Wind bracing	18,644	.....	28,380	.....	18,644	.....	28,380
Diagonals and wind bracing of counter arch	4,000	.....	.....	.....	4,000		
Wind bracing of arch. Stays	6,076	.....	.....	.....	6,076		
Rods	6,992	.....	.....	.....	6,992		
Couplings and eyeplates for rods	2,000	.....	.....	.....	2,000		
Suspension rods	6,154	.....	.....	.....	6,154		
Bolts and nails	688	.....	.....	.....	688		
Rails (60 pounds per yard)	21,000	.....	.....	.....	21,000		
Joints in centre	.....	6,000	.....	.....	.....	6,000	
Total	562,924	6,000	28,380	86,000	379,924	6,000	28,380
Total weight of ones ide span	597,304 pounds.			500,300 pounds.			
	2,400 pounds per lineal foot.			2,000 pounds per lineal foot.			
2d. Centre Span: *							
Total weights	533,600	6,000	28,380	86,000	350,600	6,000	28,380
Total weight of centre span	568,000 pounds.			471,000 pounds.			
	2,272 pounds per lineal foot.			1,884 pounds per lineal foot.			
3d. Five Spans:							
Centre span	533,600	6,000	28,380	86,000	350,600	6,000	28,380
4 side spans	2,251,700	24,000	113,520	344,000	1,402,400	24,000	113,520
Castings on top of piers and abutments	.....	150,000	.....	.....	.....	150,000	
Ends of main arches	.....	30,000	.....	.....	.....	30,000	
Levers, pins and connecting rods	12,000	.....	.....	6,000	.....	.....	
Lanterns	.....	3,000	.....	.....	.....	3,000	
Total	2,797,300	213,000	141,900 ft.B.M =37,300	436,000	1,753,000	213,000	141,900 ft.B.M =37,300
Masonry (estimated from 50 feet below superstructure.)							
2 Abutments	4,910 cubic yards.			4,636 cubic yards.			
2 Piers	3,630 " "			3,630 " "			
2 Piers	1,410 " "			1,460 " "			
Total	9,950 cubic yards.			9,726 cubic yards.			

* The centre span is not a bowstring girder with chord, but purely an arch; the thrust of which is received by the piers between which it is placed. Hence it is lighter than the other spans.

Fig. 1

Fig. 2

Fig. 3.

Fig. 4

Fig. 5

Fig. 6.

Fig. 7.

Fig. 8.

Fig. 9

Fig. 10.

Fig. 11

Fig. 12.

Fig. 13.

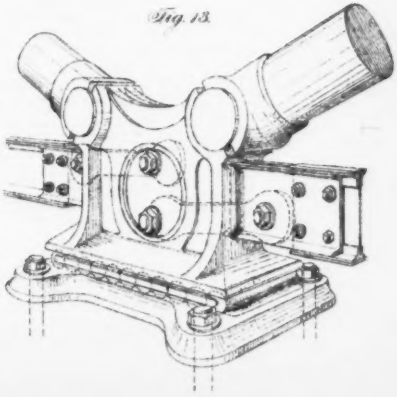


Fig. 14.

Diagram showing Strains in Braces,
of 500 feet Archbridg

Permanent Load 0 Tons per lineal foot. Movable Load 1.25 Tons per lin
by the lower Joints

Strains given for both Ribs.

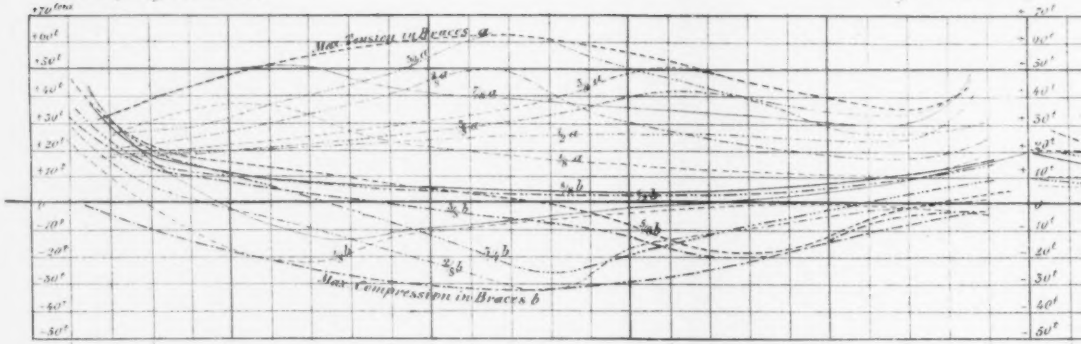
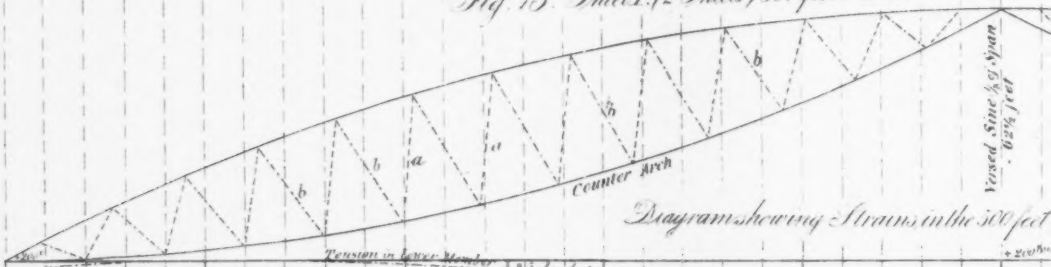
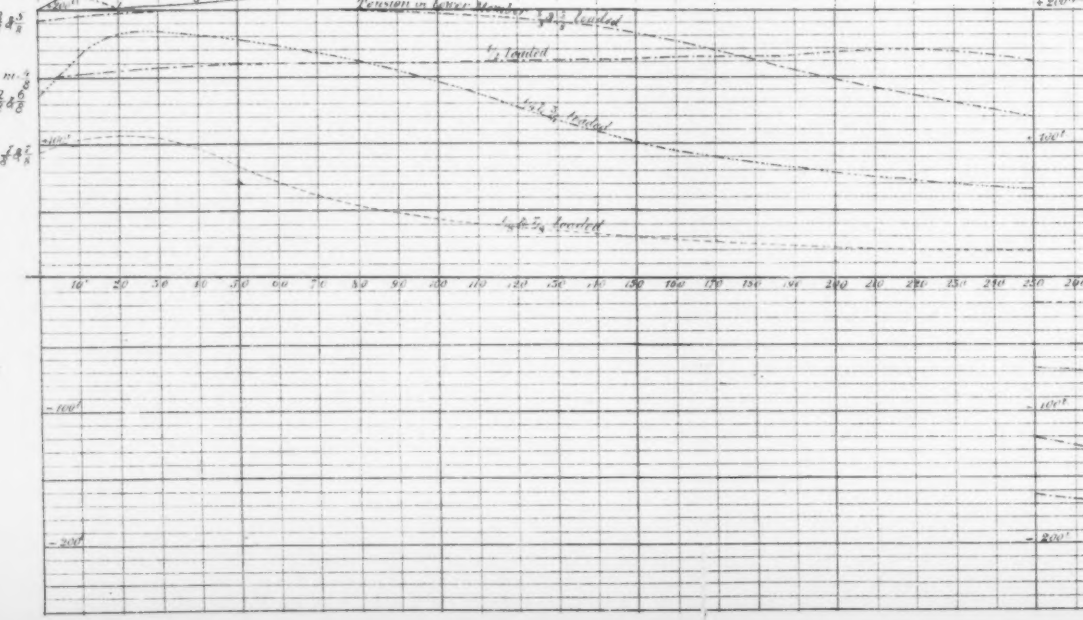


Fig. 15. Sheet 1. (2 Sheets) 500 feet Arch.

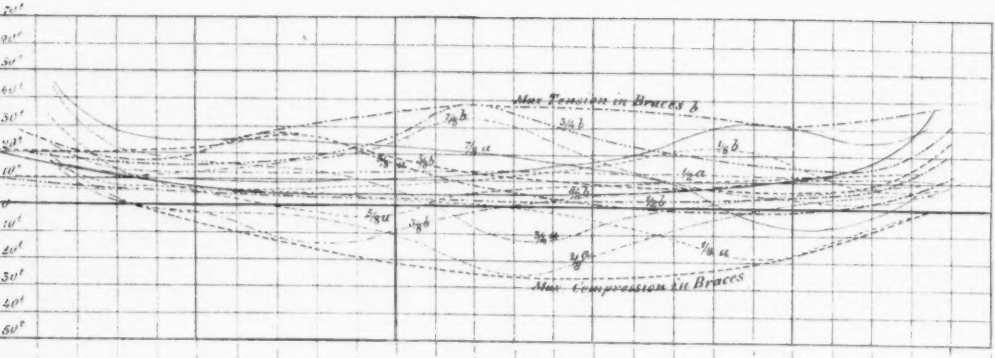


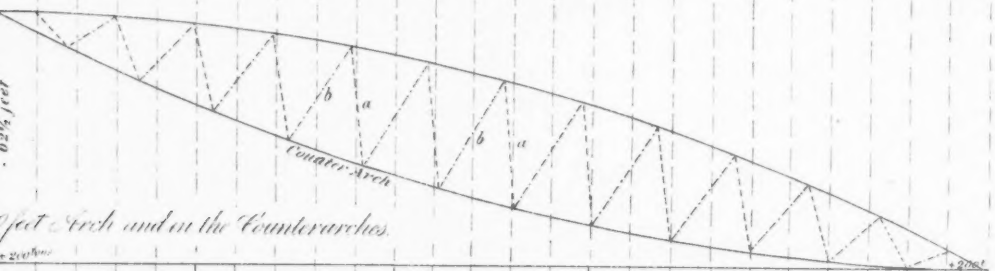


races, lower and upper Member

bridge

ns per lineal foot going from the Left to the Right supported





feet Arch and on the Counterarches

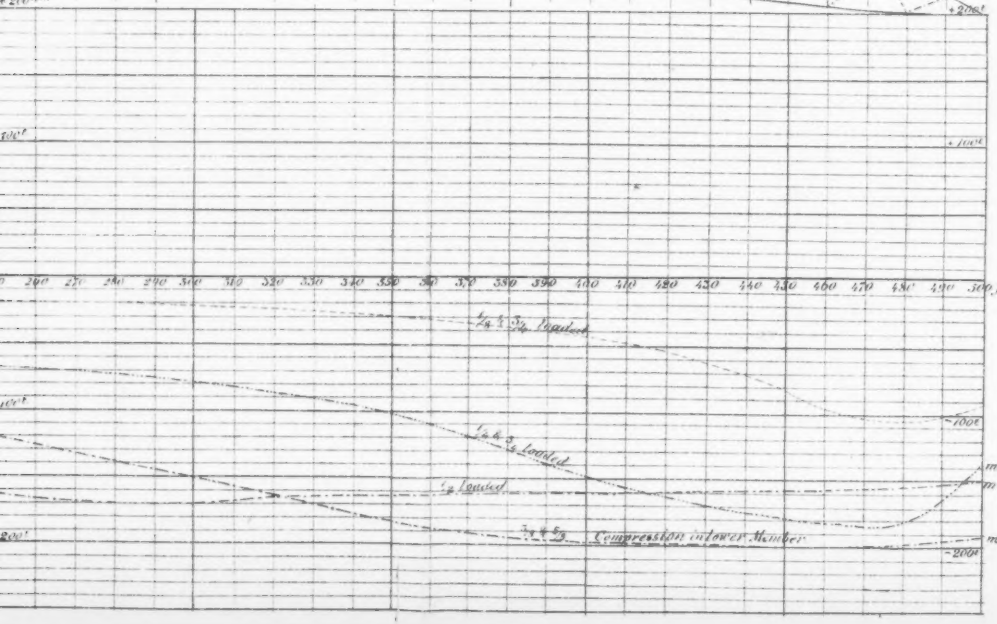
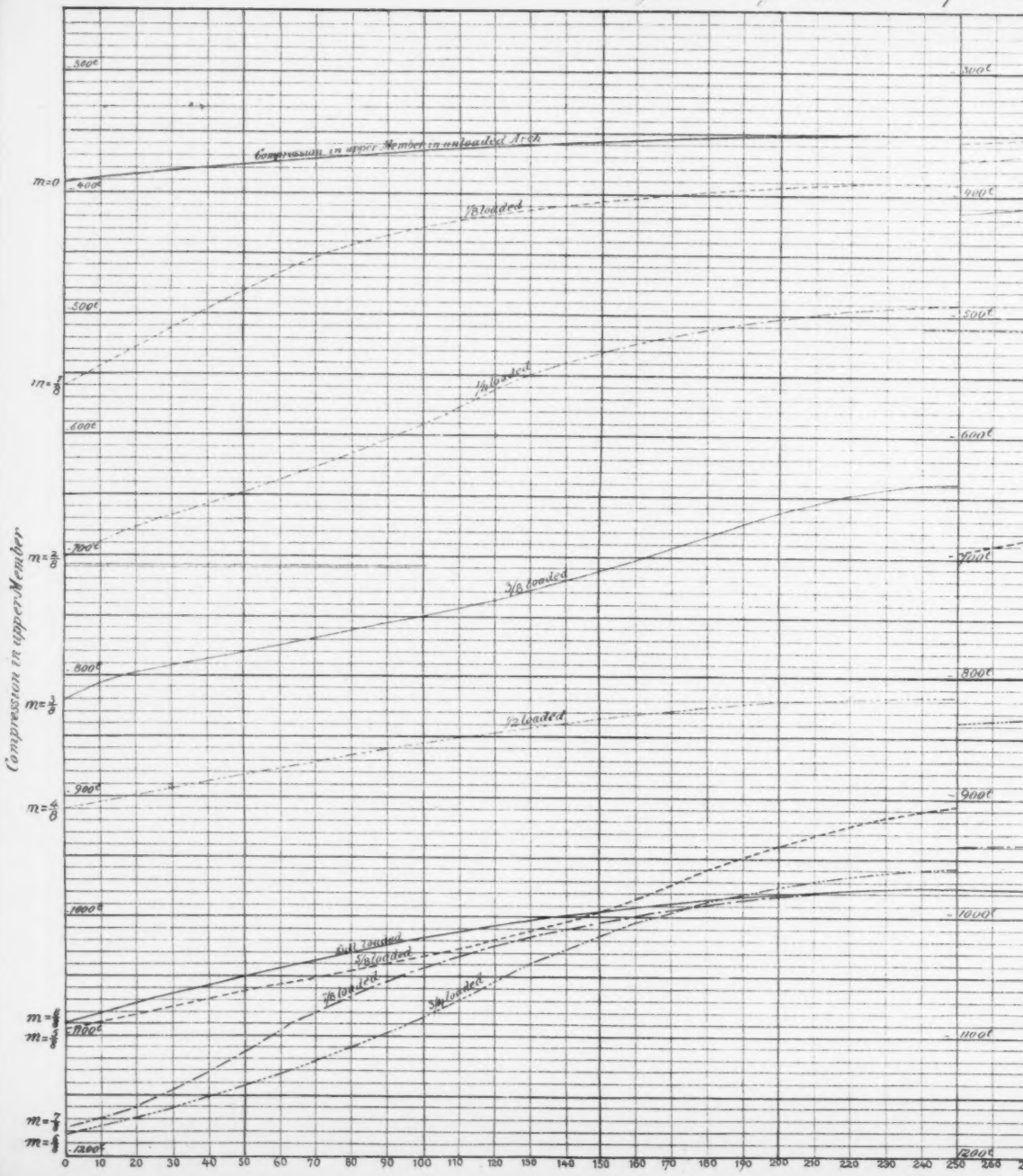


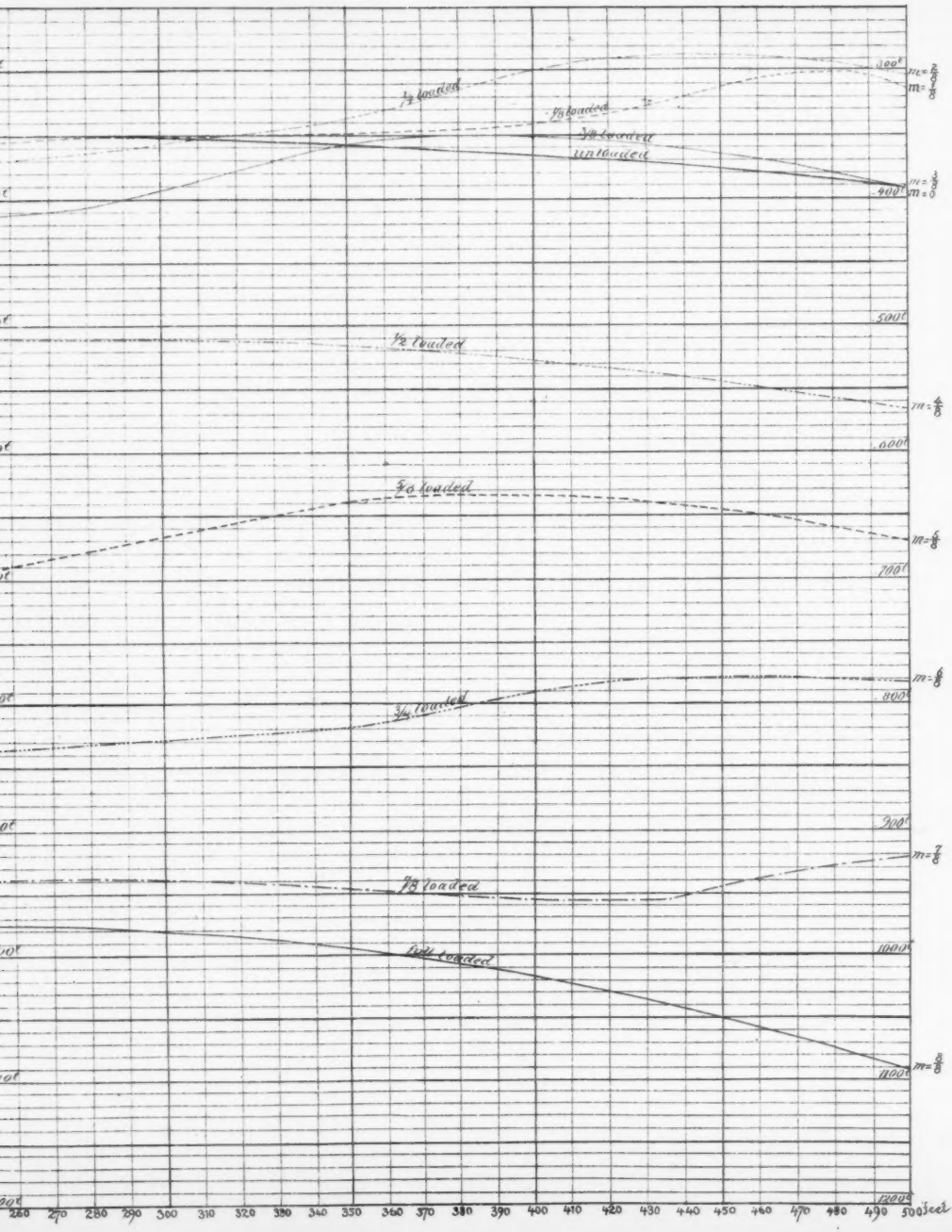
Fig. 15. Sho

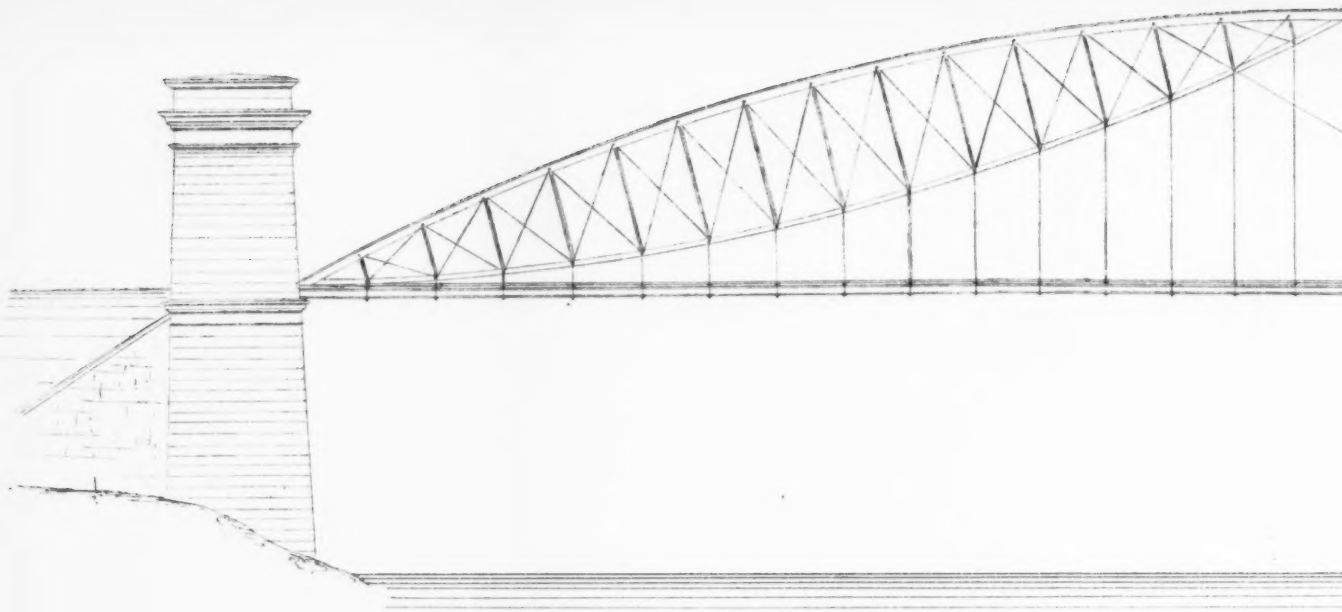
Diagram showing Strains in the 500 feet Ar



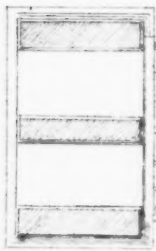
Sheet II.

Arch and in the Counterarches

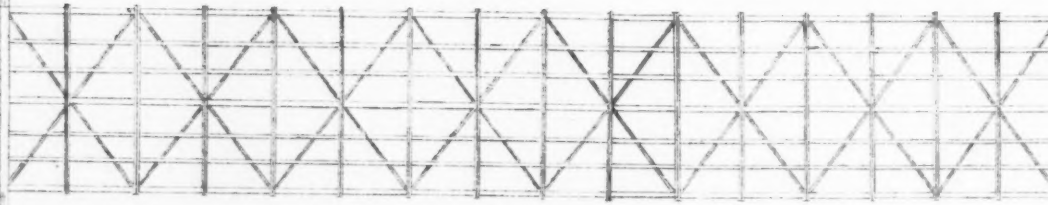




Section c-d



Windbracing of Roa



Windbracing of Main Arch

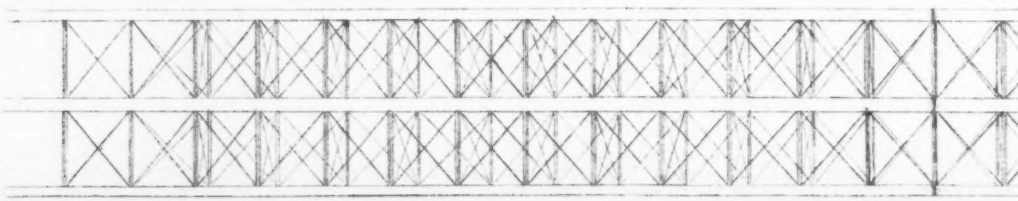
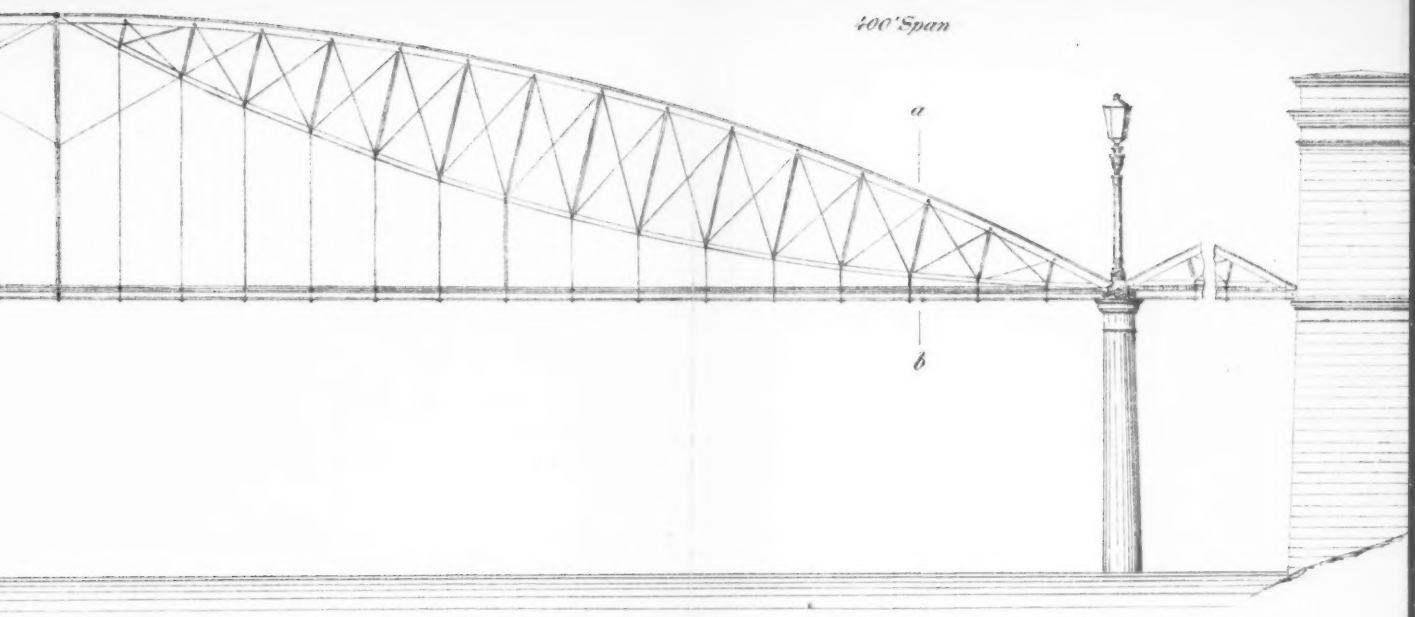


Fig 16

400' Span

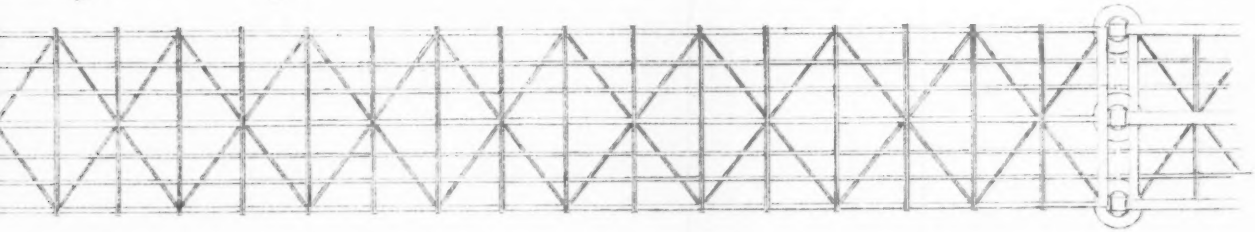
a

b



Roadway

Fig 17



Arch & Counter Arch

Fig 18

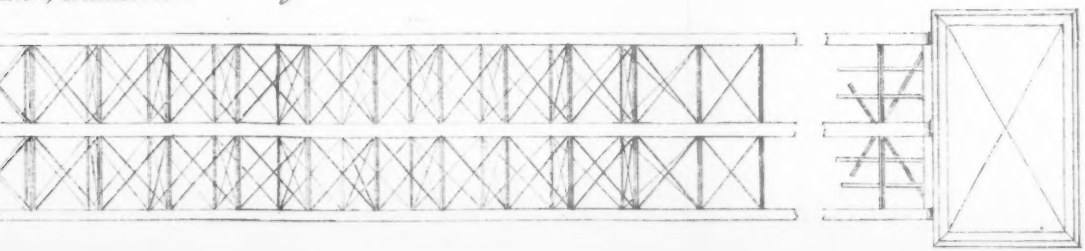
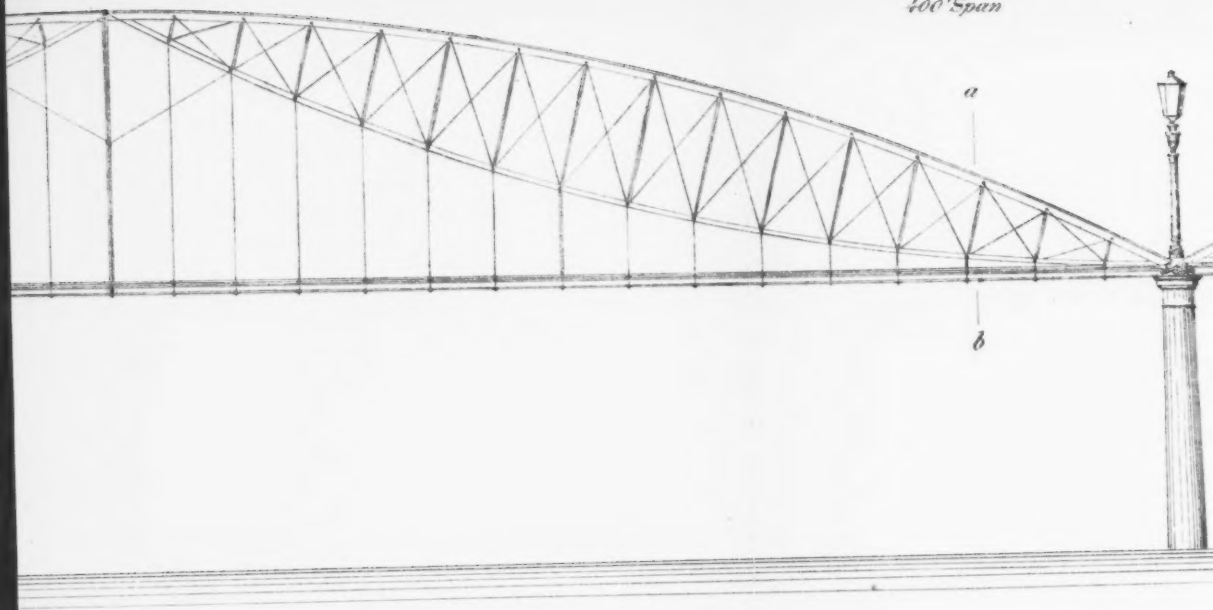


Fig. 16.

400' Span

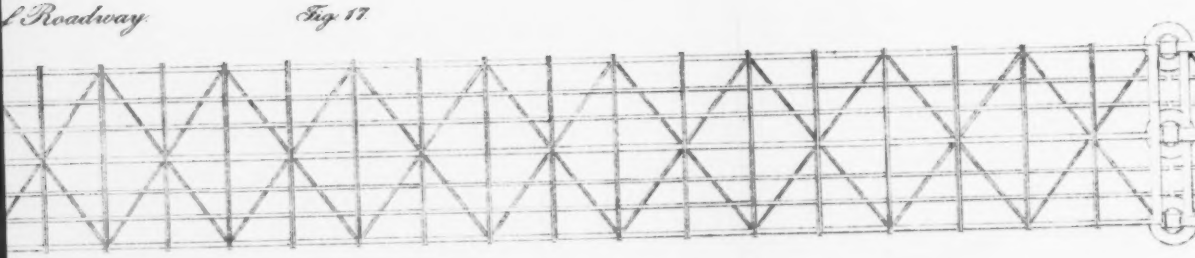
a

b



l Roadway

Fig. 17



n Arch & Counter Arm.

Fig. 18

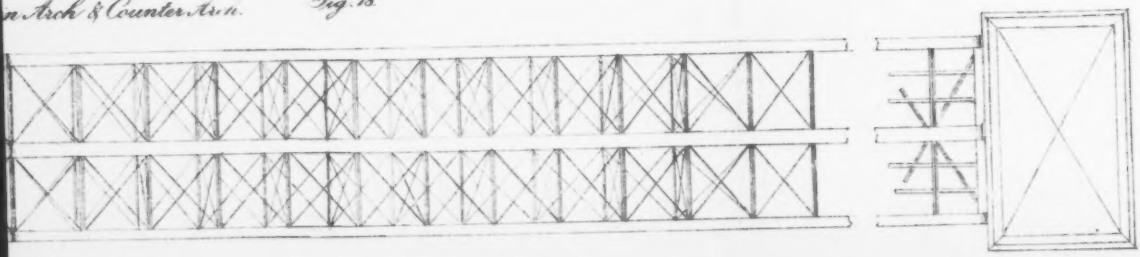
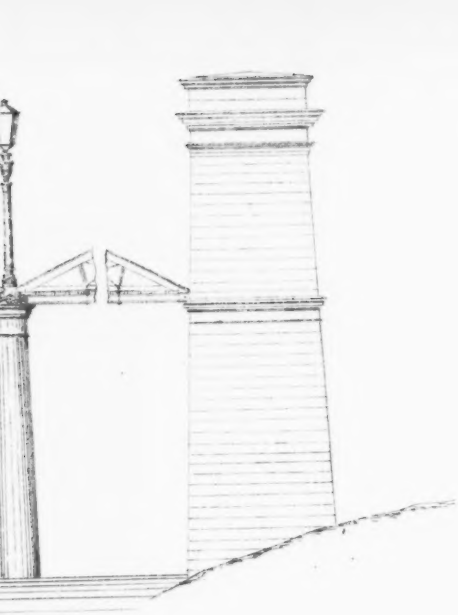
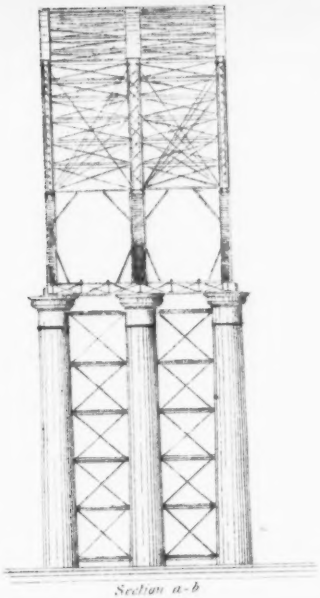


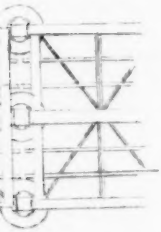
Fig. 19.

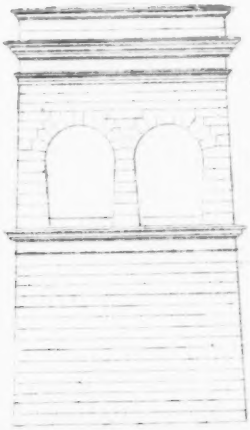




Section a-b

Fig. 20.





End View

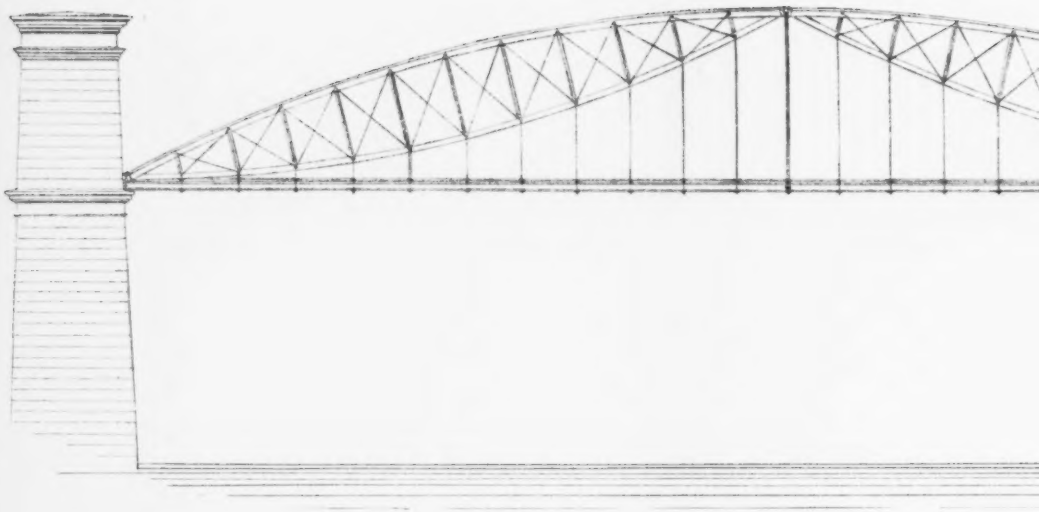
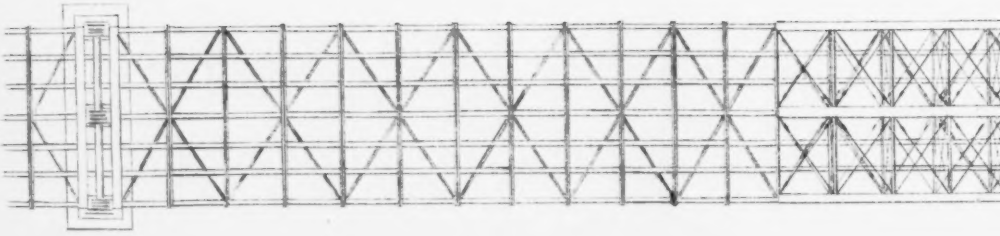
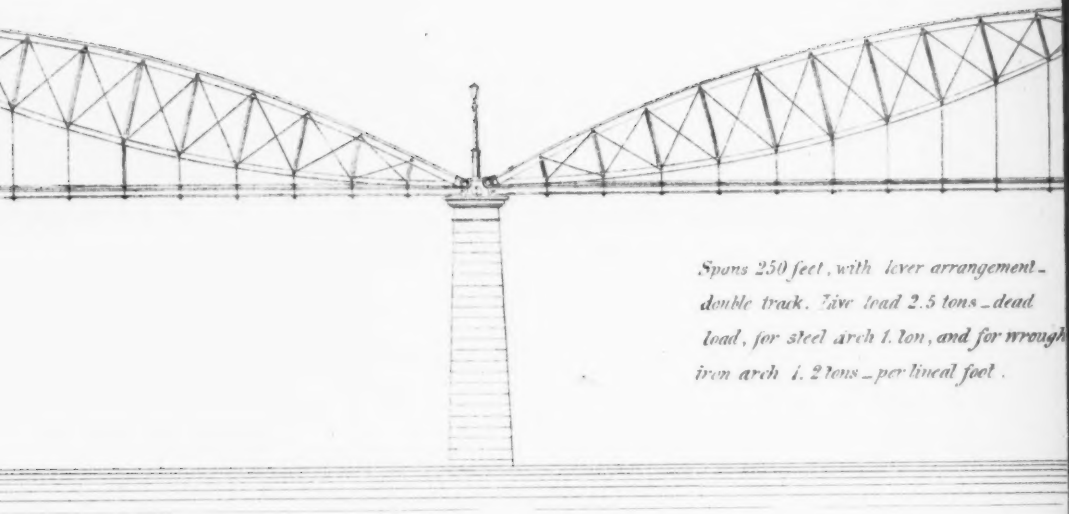


Fig 22.





Spans 250 feet, with lever arrangement - double track. Live load 2.5 tons - dead load, for steel arch 1. ton, and for wrought iron arch 1. 2 tons - per lineal foot.

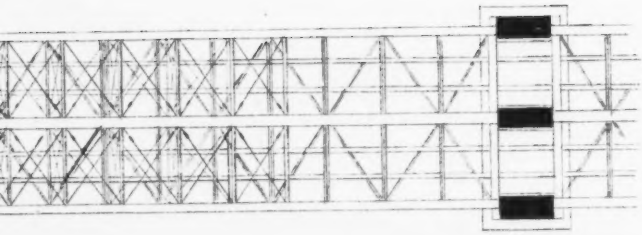
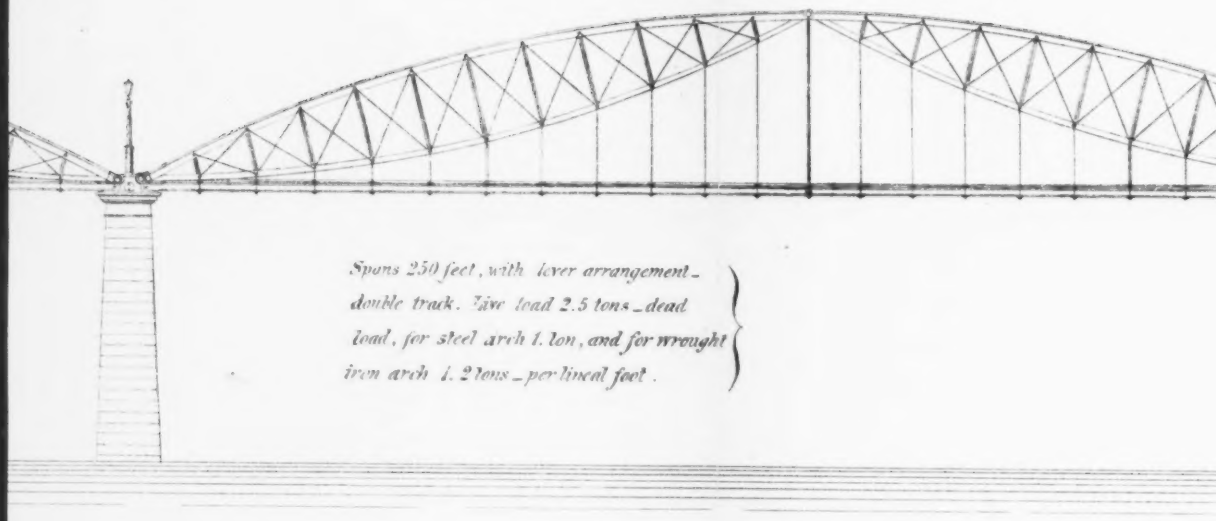


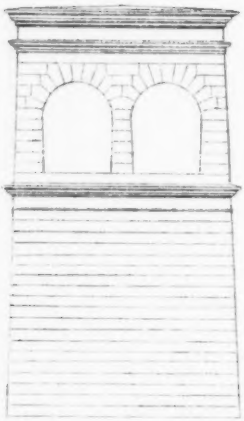
Fig. 21.

Five Spans

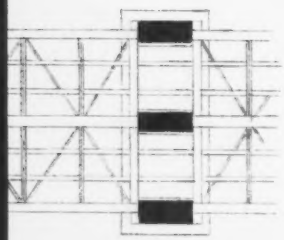


Spans 250 feet, with lever arrangement - double track. Live load 2.5 tons - dead load, for steel arch 1. ton, and for wrought iron arch 1. 2 tons - per lineal foot.

Fig. 24.



End View.



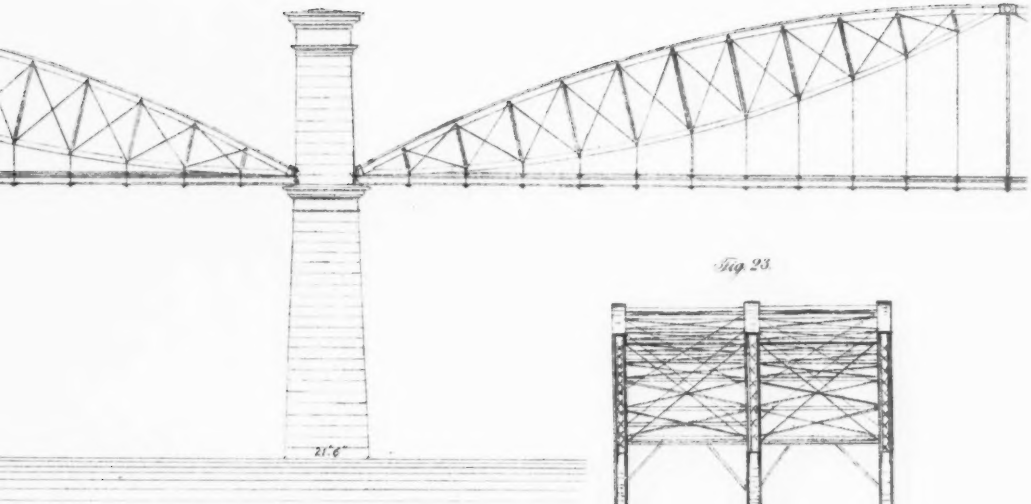
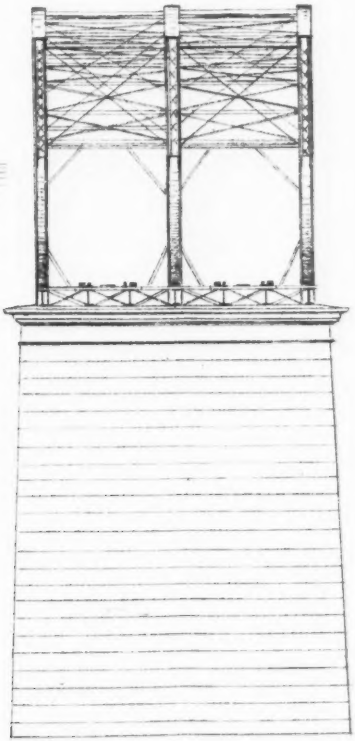


Fig 23.



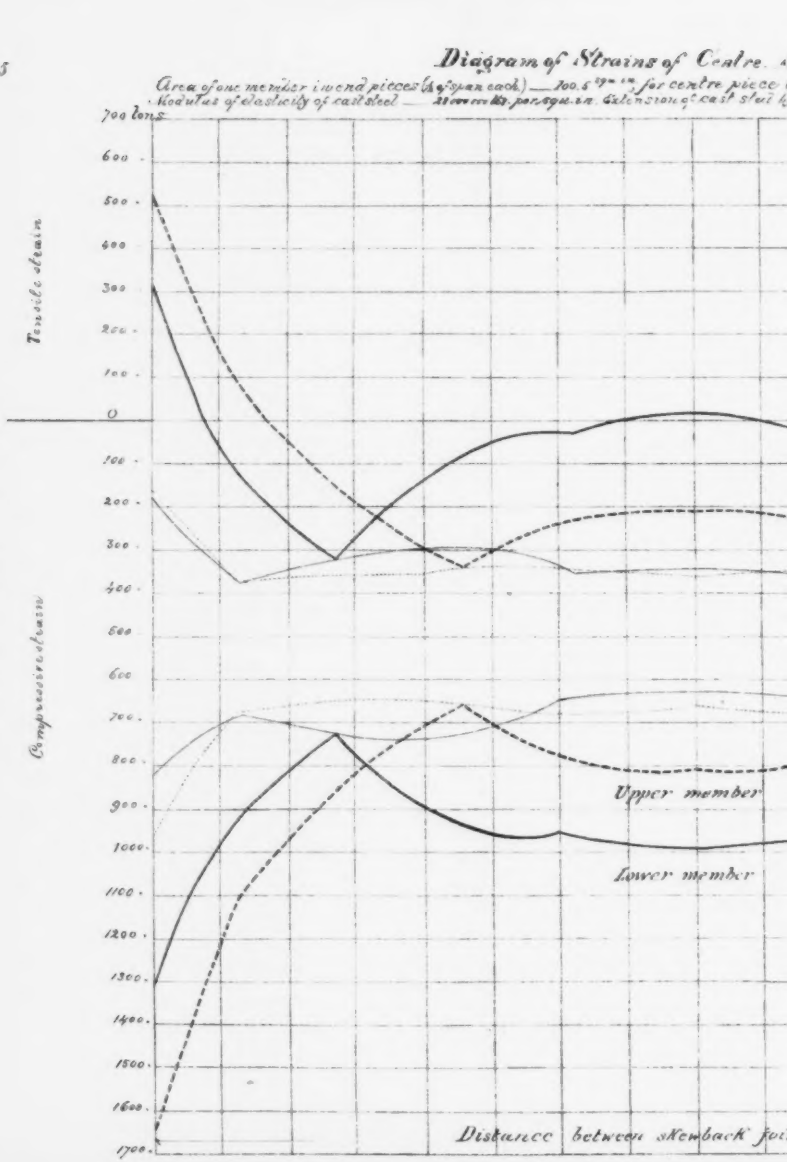
Sectional View

Fig 25

Diagram of Strains of Centre

Area of one member in end pieces (8 sq in each) ___ 100.5 sq in for centre piece

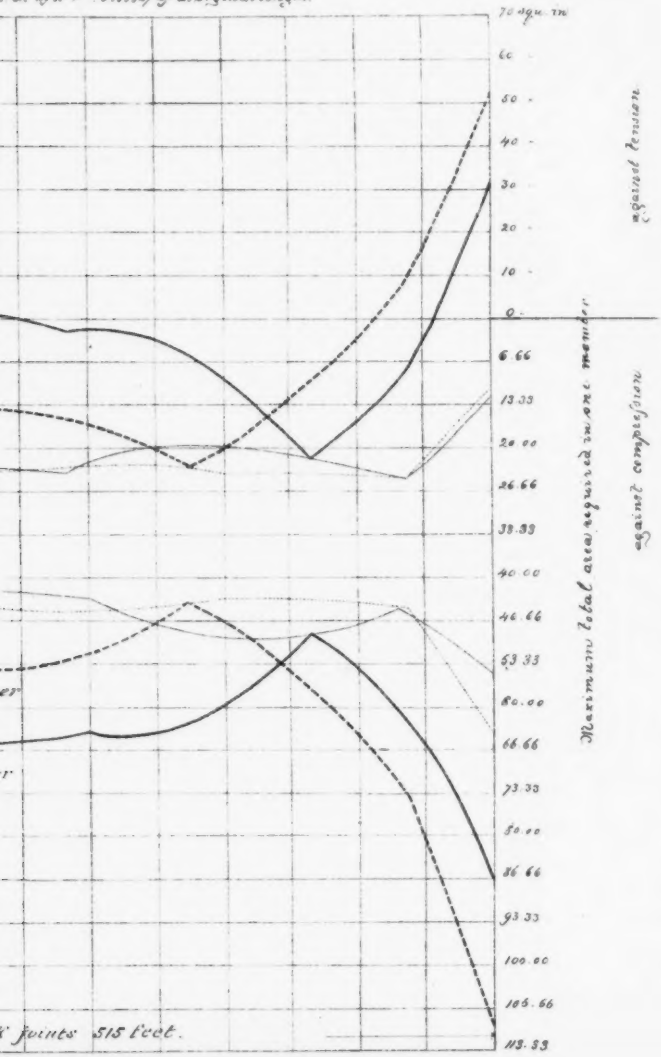
Modulus of elasticity of cast steel ___ 11 tons per sq in. Extension of cast steel



The light lines show the maximum strain or area without

re Span. Bridge at St. Louis Missouri.

iece 67 square in. Distance between the centres of upper & lower members of a rib — 12 feet.
steel by Wt. density of the original length.



joints 515 feet.

regarding the influence of temperature.

MR. SQUIRE WHIPPLE.*—With regard to this interesting paper of Capt. Eads, I desire to make the following remarks:

After recounting certain obstacles from the effects of temperature upon the arches of the St. Louis Bridge, and the means employed to obviate them, which I pass over for the present, the writer quotes Mr. Peter W. Barlow, to show that four times the amount of material required by an arch with fixed ends, to support a given load with a *given deflection*, would be required by a truss or girder. In a general point of view this conclusion seems to be essentially correct, and if in bridge structures the *utmost* rigidity were indispensable, or of paramount importance, this fact would be highly significant. But as the deflection due to elasticity of materials in a well arranged and well proportioned truss, under any degree of strain consistent with due regard to security from breakage, cannot be regarded as essentially detrimental to the value and utility of the structure, I conclude that the same factor of safety may properly be used for the same materials placed in a truss (notwithstanding its greater deflection) as for those in an arch with fixed ends. I am not aware of any practice where the amount of material has been doubled, or essentially increased, merely for diminishing deflection, and I presume the writer would not recommend it. Therefore, when we concede to the arch without chord, whatever advantage may be gained by the substitution in place of the chord of other available means for resisting the horizontal thrust of the arch, we grant all that can be reasonably claimed for it, especially in view of the fact that the deflection due to change of temperature, in the arch with fixed ends, is considerably greater than what results from elasticity in any ordinary truss. Between extremes of temperature the St. Louis arches must rise and fall over one foot in addition to what, under the passage of loads, is due to elasticity.

The chord of an *arch truss* counterbraced between arch and chord by uprights and diagonals, resists about 39 per cent. of the strain upon the whole material of the truss; this is as estimated for a 100-foot truss, and is probably somewhat less in case of greater spans, requiring more material in the web system. Could this 39 per cent. of material all be dispensed with, it would be an immense saving. But in case of unequal load the diagonals of the web system act (or should act) horizontally upon the chord, and hence the whole chord can not be done without,

* Presented October 10th, 1874, in continuation of the discussion on "Upright Arched Bridges."

even though the arch thrust be sustained by abutments under uniform load. I will assume one quarter of the usual chord section (whether used as chord or as "counter-arches" in the manner proposed by Capt. Eads) is required to act against diagonals in the web system, leaving, say 30 per cent. of the bowstring truss as ordinarily constructed, to be saved by other means provided to sustain the horizontal thrust of the arch.

With wooden arches, not materially affected by change of temperature, it could seem then that something like this quantity of *truss* material could be saved by abutments so arranged as to sustain the arch thrust. This principle is made available in the "Burr" bridge, which has been extensively adopted, and for light traffic may be classed as among the best wooden bridges hitherto put in use. The small amount of moving load to which this bridge is usually subjected upon common roads, produces no sensible disturbance in the balance of thrust of opposite arches against the same pier, and if the abutments are substantial, and the timber well enclosed, the structure will endure for a great length of time. It is believed, however, that if the bridge were counterbraced between arch and chord, in the manner of the "Whipple" iron arch truss, with chord only sufficient to withstand the action of diagonals, a better result might be obtained than on the genuine "Burr" plan. But I do not propose to discuss the subject of wooden bridges at present.

The expansion and contraction of metals attending changes of temperature, seriously disturb the concert of action between the counterbracing system and the metallic arch with stationary ends, and hence the ingenious devices of Capt. Eads described in this paper intended to make the theoretical economy of chordless arches available in connection with the use of metallic arches.

From preceding considerations it seems reasonable to conclude that some 30 per cent. of the truss material of *wooden* arches can be saved by resisting the arch thrust externally, and the same may be done with metallic arches if the effects of change of temperature are removed without incurring counterbalancing disadvantages.

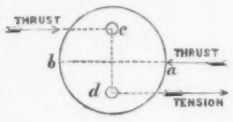
To prevent the disturbance of the web or stiffening system from the effects of rising and falling of the arch due to change of temperature, the writer proposes (the abutments being made sufficient to resist the end thrust, and the piers to sustain the weight of structure and load) that the arch be reduced to the condition of a pair of trussed rafters in the form of the "Saltash" truss, composed of erect and inverted or

"counter" arches, and connected by a web system of struts and diagonals, or of thrust and tension obliques, between erect and counter arches, the rafters being jointed at the centre and ends of the span, so the latter may remain stationary while the centre rises and falls with change of temperature. Then to enable the light piers to maintain their positions under the thrust of a loaded arch against the less thrust of an unloaded one upon the opposite side, it is proposed to introduce wooden thrust cords under the several arches, from pier to pier.

This device certainly appears feasible, but it must necessarily absorb a considerable part of the saving of metal in the chord, and the combination of materials is somewhat incongruous. Estimating this timber work and its maintenance to cost one-quarter as much as a full tension chord of iron, the apparent saving is reduced to about 20 per cent. of the cost of the arch truss with an iron chord complete. But as theoretically the *arch truss* requires from 12 to 17 per cent. more material than the best types of trapezoidal and parallel chord trusses, there can be only 3 to 8 per cent. saved by substituting the metallic arch with wooden instead of full-sized iron tie chords for the parallel chord truss, and this is to be set against the extra cost of abutments, strong enough to resist the horizontal arch-thrust.

As an alternative to the use of the wooden strut chords for the same object, Captain Eads describes an ingenious arrangement of lever discs in the skew-back block, with small thrust and tension chords, whereby only one quarter of the thrust of the arch (in case the live and dead loads are equal) is to be sustained by the iron chord. But this small chord must be so constructed as to act both by tension and compression alternately, and secured in line at short intervals, to prevent buckling under compression; hence it necessarily will be more expense than a simple tension chord of the same capacity.

Furthermore, there must be more or less uncertainty as to balance of action on these disc levers, the whole diameter of the disc representing that of the centre pin, while the other pins must have large diameters, as compared with the length of leverage. To illustrate; if *a b* be the disc and *c d* the pins, while the forces act in the direction of the arrows, the contraction of chords causes the disc to revolve, so that the bearing at *a* is transferred more or less downward, and that upon the pins *c* and *d* upward, thus shortening the lower and lengthening the upper



arm, whereby the tension acting upon the lower pin must exceed the thrust upon the upper one, possibly to a considerable extent, and when the chords expand this will be reversed.

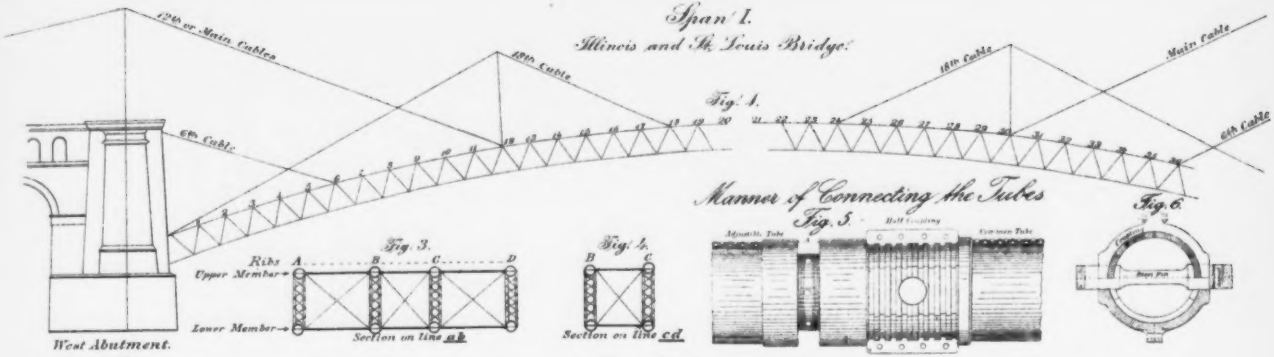
Under all these conditions and liabilities, it is not unreasonable to assume that the chord service under this arrangement would cost one-half as much as a simple tie chord to sustain the whole horizontal thrust of the arch. But taking it at only one-third of the 39 per cent., being 13 per cent. of the whole material of the complete bowstring truss, in addition to the 9 per cent. before assumed for chord or counter arch, to withstand the action of diagonals in the web system we have only 17 per cent. of the truss material of the bowstring truss saved by this arrangement, which is but a trifle, if any more, than the theoretical advantage of the parallel chord over the counterbraced bowstring truss, and the extra cost of abutments required to withstand the horizontal thrust is yet to be accounted for in the comparison.

On the whole, therefore, according to these cursory general observations, it is not very clear that there is any great economy in the substitution of the arch plan with Capt. Eads' truly ingenious devices for saving chord material and obviating the effects of change of temperature, for the excellent parallel chord truss as constructed by many skillful builders at the present day. Where the amount of action (that is the stress multiplied by the length of the several members) is so nearly balanced in different plans, greater facility of manufacture and erection may decide in favor of either plan. Should the practical test of Capt. Eads' views prove more favorable than appears from the above considerations, all will feel rejoiced thereat. If there is a chance for greater economy in the construction of bridges than has yet been realized in practice, it is highly important, in these times of universal locomotion, that the fact be recognized and the advantages made publicly available.

The estimates for material required in the proposed plans, as presented in the paper under consideration, appear decidedly low, and if sustained in the practical test, the most satisfactory results may be hoped for. I shall await further developments in the premises, with much interest, if not with very sanguine expectation of any decided revolution in the practice of bridge construction.

Span I.

Illinois and St. Louis Bridge.



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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

XCIX.

NOTES ON

THE ERECTION OF THE ILLINOIS AND ST. LOUIS BRIDGE.

A Paper by THEODORE COOPER, C. E., Member of the Society.

PRESENTED JUNE 10TH, 1874.

I.—MODE OF SETTING AND ADJUSTING THE SKEW-BACKS.—The bed-plates upon which the skew-backs were to rest were bedded into the masonry during the building of the piers and abutments. Great care was taken to get them into proper position, but it was found practically impossible, under the circumstances, to adjust them with perfect accuracy. A more accurate adjustment was therefore delayed until the skew-backs and their connected tubes (first tubes of the arches) were in place. After these were hoisted into position upon the bed-plates and anchor bolts and firmly screwed down to the bed-plates, the following measurements and observations were made to ascertain the amount and direction of variation from correct position. 1st. The angular inclination of the skew-back tubes in a vertical direction was determined. 2d. The direction of the centre line of these tubes in reference to the centre of their opposing skew-backs (at other side of the span) or their horizontal deflection. 3d. A line was drawn through the pin-holes at extremities of the four tubes composing the upper or the lower set of skew-back tubes, to determine the variation in the resultant lengths of these tubes, due to variation in thickness of skew-backs or projection of the bed-plates. 4th. The vertical distances between the pins in the upper and lower set of skew-

backs, which governed the depth of the arch rib. 5th. The horizontal distances between the centres of the skew-backs. 6th. The level of the centre line of each pin-hole at extremities of the skew-back tubes. In addition to these, as additional checks upon the correctness of the adjustment, the distances of the pins in the upper skew-backs from the pins in the extremities of the lower skew-back tubes—the length of first main braces—were taken; also the vertical and horizontal distances between the centres of the pin-holes at extremities of the skew-back tubes.

From these dimensions and observations, the amount and direction of the changes necessary to bring each skew-back and its connected tube into correct position were determined. The changes necessary to correct the errors shown by the 4th measurement were made by cutting out the holes through which the anchor bolts pass, and allowing the higher skew-back to drop the proper amount. The angular changes needed to correct both horizontal and vertical deviations were made by inserting between the skew-back and its bed-plate angular plates, made of an increased thickness under those skew-backs which did not project as far as the others of the same set.

These adjustments necessarily required much time and care, but considering their great importance as bearing upon the correctness of the several arch ribs which spring from these skew-backs, the time was well spent. After these changes were made, the above measurements were again repeated to determine the correctness of the adjustment, and where they were found not perfectly satisfactory, there was further adjustment.

II.—Insertion of the Central Tubes.—To make clear the difficulties to be overcome in inserting the central or "key-stone" tubes of an arch of the size and character of those of this bridge, it will be necessary to describe briefly the arches, the method of connecting the tubes, and such other conditions, as influenced the operation of uniting the semi-arches into one complete arch by entering and connecting the central tubes.

1st. Each span is composed of four arched *ribs* spaced respectively 16½, 12 and 16½ feet apart from centre to centre (Figs. 3 and 4). Each of these ribs is composed of two lines of tubes (called the upper and lower *member* according to their position), spaced 12 feet apart between centres of the tubes. The several tubes are about 12 feet long. The upper and lower members of each rib are connected together by a system of triangular bracing. The first or skew-back tubes of each member are rigidly fastened (screwed into) the large wrought-iron skew-backs, which in turn rest upon the bed-plates set into the masonry; the whole being firmly anchored to the piers or abutments by large steel anchor bolts. The

several tubes forming one line or member are rigidly connected at their ends by grooved couplings (Fig. 5), carefully fitted to grooves cut on the ends of the tubes. The couplings are made in halves with flanges for bolting them together. Through the centre of each coupling and the ends of the tubes passes a taper pin-hole for the steel pin, upon which the braces and cross-stays are attached. All joints were made as nearly perfect as the best mechanical appliances would permit.

2d. The tube being erected, was hoisted into its place, and the end to be connected brought into exact abutting condition with the end of the tube already in position. The half coupling containing the larger pin-hole (the pin being taper as before mentioned) was then forced upon the grooves of the tubes; the pin was entered but not driven home—the remaining half coupling was put in place; then the coupling halves were drawn together by their bolts and blows of mauls, until they came to a solid bearing upon the grooves of the tubes; this being readily determined, in case of doubt, by removing the pin and examining the joints of the grooves. Before the couplings were very tight upon the grooves, the pin was driven into its position to draw the tubes into correct line circumferentially. The tubes of the lower members were always connected first, and afterwards the tubes of the upper members. After the lower and upper tubes of one section were in position, the main braces of the preceding section were put on. In order to get these over the pins it was necessary to force the members apart by jacks, or draw them together with tackle, according to the inclination of the braces—those inclining from the lower pins towards the pier of that half span requiring the tackle, and those towards the centre of the span, the jack.

In order, therefore, to connect a new tube to those already in position, it was essential that the faces of the two tubes should abut accurately on all sides and the pin-holes be in correct correspondence. This was comparatively a simple operation when only one end of a tube was to be connected; but when it became necessary to connect a single tube at both ends,—as would be the case when the central tube was to be connected to the projecting ends of the tubes of two opposing semi-ribs—the difficulties became much greater.

3d. The method of sustaining the semi-ribs during their erection was that of suspending them from cables leading from temporary piers or towers on the permanent piers of the bridge, as designed by Col. Henry Flad, of St. Louis.* A cable was attached to the ribs at every

* See "Description of the proposed Plan for Erecting the Superstructure of the Illinois and St. Louis Bridge," by Walter Katte, C. E. Transactions XLVII.

third joint of the span. These cables took their names from the number of the joint to which they were attached, counting from each end of the span :—as cables No. 3, No. 6, No. 9, No. 12 (main cables), No. 15 and No. 18. At no time were more than three cables in the same semi-rib under strain together, and generally only two. The sixth, twelfth and eighteenth cables were the permanent ones (during the erection) and the third, ninth and fifteenth, the auxiliary cables ; the third being removed when the ninth was under strain, the ninth when the twelfth, and the fifteenth when the eighteenth was under strain.

The twelfth or main cables were strained and balanced by hydraulic rams beneath the main towers; the fixed strain being maintained during all changes of temperature by means of a weighted balance-gauge connecting with the rams. All other cables were strained by adjustable screws, and had no adjustment for temperature; the variations by temperature in these cables being but little different from those in the length of arch controlled by them, it was unnecessary.

4th. Each of these cables was strained to a certain amount for each different condition of the erected portion of the arch, with the object of restricting the tensile strain upon the steel couplings connecting the tubes of the lower members of the ribs within 40 tons for each coupling; this being a proviso insisted upon by the contractors, although every steel coupling in the bridge had been actually tested to a strain of 300 tons and over. The strains in the main cables were measured by the weights upon the automatic balance gauge which operated the hydraulic rams supporting the main towers. The strains in the other cables were determined by a micrometer measurement of the extension of the links, —one set of links in each cable having been previously marked for this purpose.

5th. To compensate for the decrease of length of the tubes due to the compression under the weight and load of the finished spans, each tube was made longer than the length calculated from the normal curve of the arch by an amount equal to this loss from compression ; or, in other words, the tubes were calculated for a span longer than the actual distance between the abutments. By this, the line of tubes for the side spans, without any compression, would measure 3.2 inches longer than they would when under their mean compression in the finished work, or each semi-rib might be considered as 1.6 inches too long.

6th. The long semi-arches of steel were very sensitive to the action of the temperature, being not only subject to direct extension and contraction, by heat or cold, but also to a curving action when one rib became

more or less heated than the other. The bridge standing nearly east and west, one side usually received the most direct sun heat.

7th. Starting from the skew-back tubes, all four ribs were simultaneously erected as far as the tenth or eleventh tubes (Fig. 1). At this point the outer tubes were discontinued, and only the two inner ribs carried to the centre. The several ribs were connected and stiffened by their connecting struts and tension rods. At the centre panel, these long semi-arches presented a rectangular cross-section (Section *c d*, Figs. 2 and 4) about 12 feet deep and 12 feet wide. Considering that each line of tubes, after being united into a complete arch by the central tube would be subjected to compressive and tensile strains, according to the action of the temperature, it was absolutely necessary that both lines of tubes on the same horizontal line, either upper or lower members, should be joined at the same time, to prevent the buckling or warping of the ribs; and not to permit one central tube to become jammed by an increase of temperature or other cause, before the corresponding tube in the adjacent rib should be ready to take its share of the compression.

8th. The standard of temperature to which the computation and measurements were referred is 60° Fah., and the curve of the finished arch, under its permanent load at this temperature, was taken as the "normal curve of the arch."

To enter the central tubes into their place between the projecting ends of the semi-ribs of these arches, the increased length of these semi-ribs due to the construction as mentioned (5th), had first to be overcome. This was partly accomplished by the compression due to the cable-strains; the remainder, it was expected, could be reduced either by increasing the elevation of the arches by means of the cables, by taking advantage of a low temperature, or by a combination of the two. From the cable-strains being restricted, as mentioned (4th), the stiffer ribs (and it was found that they varied in rigidity) would, of necessity, be lower in position than those which were more flexible. A rib which was but a little low at the first cables would show a rapidly increasing declension as it extended towards the centre, and after a rib was once allowed to get low, it would require a much higher cable-strain to bring it up again than would be necessary to equilibrate the same had it been kept at its elevation during the erection.

The insertion of these tubes in the first span was, of course, the most difficult, and occupied the most time. Every step had to be performed with caution in order to study its effects. Many preconceived plans had to be altered or rejected entirely, as experiments proved them to be

wrong. For the first span, by order of the Chief Engineer, Capt. James B. Eads, two sets of tubes were prepared; one similar in construction to those composing the arches, and made of the exact calculated length of the central space; the other differing from these only in that they were fitted with an internal adjustable screw, by means of which the tubes could be lengthened or shortened to a limited extent, in case there should be found any errors in the measurements of the span. Both tubes were of equal strength; those, however, fitted with an adjustable screw presented a different external appearance from the ordinary tubes, due to the extra banding necessary at the juncture of the two parts forming these tubes. (Figs. 5 and 6.)

Span I.—The last tubes necessary to complete the two inner ribs of this span up to the central tubes were erected on September 5th, 1873. The expense and responsibility of the insertion of the central connecting tubes of the arches devolved by contract upon the Illinois & St. Louis Bridge Co.; under the direction of Col. Henry Flad, acting Chief Engineer (Capt. Eads being absent in England), the writer was given charge of this work; the men were furnished by the Keystone Bridge Co.

At this date the semi-arches were supported by three sets of cables at each half of span, viz., at the sixth, twelfth and eighteenth joints. The sixth cables were attached to all four ribs, and contained a cross section of 13 square inches for each rib. The twelfth or main cables were attached to the two inner ribs only (the outer ribs terminated at the eleventh joints), and contained a sectional area of 42 square inches for each rib. The eighteenth cables, also attached to inner ribs only, contained a sectional area of 22 square inches for each rib. The sixth cables at this date were under a strain of 65 tons, the main cables 176 tons, and the eighteenth cables 73 tons per rib.

The arches having been erected without reference to their elevations or correctness of line in direction of the span, stood in the following condition; the western half of the span was one foot lower in elevation (measured at the extremities of the ribs) than the opposite or eastern half; the western half being 8¾ inches below the normal line of the finished arch, and the eastern 1¼ inches above, the remaining 1½ inches being the difference in elevation of the ends of the central tube (its higher end being the *true* centre of the arch). Horizontally both semi-ribs were curved to the north, the western 3 inches and the eastern one inch, measured at the extremities of the ribs. This curvature was due to the tension-rods running in one direction towards the centre, being

more easily screwed up than those in the other direction; one set being fitted with sleeve nuts and the others with swivels. The ribs being in this condition, a measurement of the space between the two opposing semi-arches at a temperature of 60° Fah., showed that the upper members of the ribs were too close together by 0.3 inch, and the lower members by 0.5 inch. This was considered at the time very satisfactory, as it was not doubted, but that with an increase of cable-strains and a lower temperature the connecting tubes could be readily entered.

A few days were spent in throwing the arches into line horizontally, stiffening the ribs with their diagonal rods, riveting the lattice straps between the main brace bars and in other work necessary to prepare the ribs for the new strains, which would be thrown upon them by temperature as soon as they were connected rigidly together.

During this time experiments were made upon the action of the ribs under increase and variation of cable strains, the ultimate object being to raise the extremity of the western half to a corresponding elevation with the eastern. The results as obtained from this semi-arch were not very satisfactory for the purpose intended, for the arch was found to be very stiff and slow to yield under any increase of strain. From the great mass of metal to be moved, it was very important to study carefully the action of each increase of strain upon the arches and their cable system. Several days were, therefore, spent in these operations. After straining the eighteenth cable as greatly as was considered proper (to about 10,000 pounds per square inch), the main cables to 200 tons, and slacking off entirely the sixth cables from the inner ribs, this semi-arch, on September 14th, was 7 inches higher than it was on the 5th, making its half of the span still 3½ inches too low as compared with the eastern half.

At 7 o'clock A. M., this date, the temperature being 44° Fah., the space between the opposing ribs was found to be in excess of the required distance, ⅜-inch for upper members and ¼-inch for lower. These measurements, however, did not give the true amount of clearance which the tubes required, as the additional space needed to enter the tubes in an oblique direction, which the want of level of the two opposing ribs made necessary, was not taken into account.

As it was not considered desirable to strain the cables beyond the amount they were then subjected to, it was determined to try to enter the lower tubes in this condition of the arches. These once entered, the two semi-arches could readily have been brought to a corresponding level without any risk, by decreasing one set of cable strains and increas-

ing the other. Having secured the connection of the lower tubes, it was believed there would be but little difficulty in entering the upper tubes on an increase of the temperature.

At 9 o'clock A. M., the plain tubes (not adjustable) were raised from the flat boat to the arches. The north lower tube (*C* rib, Figs. 3 and 4) entered readily into its place; the south one (*B* rib), however, would only enter part way. Attempts were made to force it into position, but the risk of getting it caught part way down was too great, as the sun was already expanding this side of the arch. Both tubes, therefore, were removed, and the insertion postponed until the next morning, at 5 o'clock. The following night was less cool, and at the hour named, the temperature being 53° Fah., the space was too short by one-sixteenth inch.

Hoping to gain this small amount, an additional strain of 6 tons was added to the main cables of the western side, but without any immediate effect. Believing that the distance saved by having the eastern side higher than the western, was more than balanced by the loss due to the faces of the projecting tubes being one higher than the other, the eastern half was lowered to a corresponding level with the western. For several days the temperature, especially at night, had been quite cool; it now became very warm, and, judging from the weather "probabilities," no immediate hope of a change to cooler could be indulged.

The Directors of the Bridge Company having made some agreement dependent upon this span being closed by September 19th, Colonel Flad determined to attempt a reduction of the arches by means of ice, in order to enter these tubes; not wishing to resort to the adjustable tubes until all chances of inserting the others were tried. For this purpose, all the tubes of these two inner ribs, over both halves of the span, were wrapped with gunny cloth. Boards, on edge, were placed between the braces on each side of the tubes, thus forming continuous lines of troughs, in which ice could be placed and the tubes cooled. At sunset of the 15th the arches were too close together by $2\frac{1}{4}$ inches—that is, the tubes would not enter by this amount. The temperature had been quite warm, about 80° Fah. in the shade. Preparations for using the ice were not completed until the 16th, at 2 o'clock A. M.; 10 tons of ice were broken, hoisted, and distributed over the tubes, the bagging being first thoroughly wetted with a hose. During the whole night a warm southerly wind prevailed, and, with the temperature above 60° Fah., counteracted much of the effect of the ice. At sunrise the space was still $\frac{3}{4}$-inch too short, but the ice had produced a contraction since sunset the day before, equivalent to a reduction of temperature of 42°.

During the day and all night of the 16th, the icing was continued, in hopes to finally overcome the influence of the warm weather. By evening the space had become reduced, so that to enter the tubes a contraction of 1½ inches was needed. At 11½ o'clock P. M., this was decreased to ¾-inch, at which it remained until the morning of the 17th. This night, also, a very warm southerly wind prevailed, and cut away the ice as fast as 50 men could hoist and distribute it.

On the morning of the 17th, there being no prospects of a change to colder weather within the time to which we were limited, Col. Flad felt compelled to order the use of the extensible tubes; especially as these were only fitted with a moderate amount of adjustment (about 1½ inches), so that, should the weather continue as it was, they even could not be entered after the effects of the ice had disappeared. The lower of these tubes were first entered; there was some little difficulty with the south one, but finally both were in place, and one end of each tube was readily connected by means of its coupling and steel pin. Before the other ends could be connected it was necessary, by means of lines and clamps, to draw the tubes into correct line with the projecting tubes of the opposing ribs. The steel pins belonging to these joints were then entered into the pin-holes of the tubes, the coupling being left off for the time; the pins held the tube ends in line vertically, and the screw clamps kept them from moving horizontally. Then the bearing faces were brought together by means of the adjustable screws and a slight lowering of the eastern ribs by slacking off the main cables. The couplings were next readily entered upon the grooves of the tubes, and the connection of the lower members completed. Having, by means of struts and rods, secured the lower members against any tendency to buckle horizontally, the upper tubes were entered in a similar manner. The braces connecting the upper and lower members were all put on, (excepting one set, which could not be until the adjustable tubes were screwed out to proper lengths), and the whole firmly strutted and tied together. At 10 o'clock P.M., September 17th, after 65 hours' constant work with a force of about 60 men, the first arch was successfully connected.

From the difficulties thus experienced, it was decided to use only adjustable tubes for the remaining spans; it being too important a matter to be dependent upon the condition of the weather and the uncertain action of the arches under the cable strains. After experience in screwing out the arches to their proper lengths, as will be explained hereafter, a change in the manner of connecting the last joints was adopted. After

the central tubes were rigidly connected at one end by means of the coupling and pin, the remaining end was only connected by a loose pin joint, as follows:—the tube ends were brought into an abutting condition, the steel pin entered into the pin-hole through the tube ends, and the tubes prevented from moving sidewise by means of clamps. As in this method the couplings were left off from the joint, much trouble was avoided. These joints were kept in this condition until the central tubes were screwed out to their proper length, when the permanent connections by means of the couplings were made. This method was perfectly safe so long as the cables were under strain and the tube ends kept at a true bearing with each other.

Span II.—Both sides of this span, came together well as to level; the west half was perfectly straight for centre line; the eastern half was bowed to the south 1¼ inches at its extremity; on Dec. 16th this arch was readily connected in a few hours, the temperature and elevation being very favorable.

Span III.—This last span gave considerable trouble, as the circumstances were very similar to those of Span I; one side (the eastern in this case) being much lower and stiffer than its opposing semi-arch; the weather was quite warm for the season (58° Fah.), and, also, it was financially necessary to close the span by a certain time. This latter fact was not known till December 17th, at noon, and the time expired on the morning of the 18th. At the first-named hour (when I received orders from the Chief Engineer, Capt. Eads, to have this span closed by 7 o'clock the next morning) the eastern half was still 3 inches too low, the main cables were strained to 200 tons per rib, and the others to their full capacity. The tubes, after being shortened as much as the adjustable screws would allow in their present condition of thread, were yet too long by ½-inch; the hydraulic jacks of the east abutment towers, by means of which the main cables for this half span were strained, had reached their full throw from the strain they were under and the unusual warmth of the season (they having been lowered some time before on account of the cold weather).

In order to succeed in entering the tubes, it became necessary to raise the jacks upon blocking, and to increase the strain on the main cables of the east side to 210 tons. The screws of the adjustable tubes were also cut by filing and chipping so as to obtain ¾-inch more adjustment. At 10 o'clock P. M., the wind, which had been quite warm, changed to the north and produced a sufficient reduction of temperature to assure the entering of the tubes, and at 11½ o'clock P. M., every preparation being completed, the tubes were hoisted to the arches. The remainder

of the work was merely a repetition of previous experience, but was necessarily delayed by the inconveniences of working upon scaffolding by torchlight. At 7.40 o'clock A. M., on the 18th, the last arch was closed.

As soon as each span was connected by the insertion of the central tubes of the inner ribs as just described, the erection of the remaining tubes of the outer ribs was continued. With the exception of cables Nos. 3 and 6, which were used for supporting the first eleven tubes during their erection, no cables were used for these outer ribs. At the time of connecting the inner ribs, those outer ribs were sustained by a cable at the sixth joint only—the third cable having been removed at the same time it had been removed from the inner ribs.

During the continuation of the erection of these outer ribs beyond the eleventh joints, the weights of these outer ribs were transferred to the inner ribs by means of temporary diagonal rods attached to the tubes by clamps. The rods used for this purpose were the lateral rods belonging to this part of the work. One of these rods was attached to each length of tube, and a double set at each third tube. As each rod was fitted with an adjusting screw, it was very easy to keep these outer ribs at any desired elevation with reference to the inner ribs to which they were attached. No difficulty was therefore experienced in inserting their connecting central tubes.

III.—Screwing out the Arches.—After the connection of the inner ribs of the first span by the use of the adjustable tubes, it became necessary to screw out these tubes to their full length as required by the calculations. The screws in these tubes were made from forged wrought-iron rings 18 inches long, 15 inches outside and 11 inches inside diameter. The screw thread was right and left-handed on the different ends, square and $\frac{3}{10}$-inch deep—the pitch was $\frac{3}{4}$-inch. In the cylindrical part of the screw, between the right and left threads, six holes $1\frac{1}{2}$ inches in diameter were bored, for the purpose of passing rivets to fasten the steel rings, which were to fill space *A* (Fig. 5) after the tubes were screwed out to proper length, and make a solid steel connection throughout the tube for compression. The screws had been prepared of this form, with the original expectation that they would only be needed to lengthen or shorten the tubes before they were inserted into their place; the necessity of forcing apart the arches with these screws had not been anticipated. They fitted very snugly and required a moderate power to turn them, even without any pressure against the threads; and with any strain of either compression or tension upon the tubes it became very difficult work to move them by the use of bars inserted into the $1\frac{1}{2}$-inch holes.

After the inner ribs were connected the cable strains were reduced so that there was only about 160 tons strain on the main cables. Then the erection of the outer ribs was going on, and this threw the additional weight of these outer ribs upon the inner ribs.

Efforts were made to turn out the screws in the first arch by means of bars inserted in the holes of each screw, connected at the ends in a wheel form. But slow and slight progress was made, as the bars were constantly being broken. The tubes being fastened at both ends by means of their couplings, were subject to the strains of compression and tension according as the temperature varied. Only at the interval when these strains were zero could any result be obtained in moving the screws. With the varying temperatures, to determine when this occurred was only possible by keeping a constant strain upon the bars. The great labor necessary and slow progress made, led to two improvements which overcame the difficulty in the remaining parts of the arches. First, the remaining ribs were not rigidly connected by couplings on both ends of the central tubes; one end of these tubes was left free by making a pin-joint only, as before mentioned. By this means no tension could be got on these tubes, and any opening or relaxation of compression at this loose joint produced by contraction or expansion of the arches (according as it was a lower or upper tube) could readily be taken advantage of, by setting out the screws. The second improvement was the designing of a wrench of a suitable form to turn these screws, even under a large compressive strain. These wrenches were made of railroad iron and resembled in form an elongated letter A. Each foot was fitted with a T shaped nipper, sufficiently narrow to enter the opening (*A*, Fig. 5) between the bands of the tubes, which in some cases was only $\frac{3}{4}$-inch. This wrench was about 7 feet long and acted by direct shearing strain upon a $1\frac{1}{2}$-inch steel pin passing through the screw and projecting sufficiently at each side for the nippers to catch it. By means of the bars, as first used, we could never put a greater strain upon the screws than exerted by four men on an ordinary crab with a single line; but with the use of the wrench the power of six men on a double-geared crab could be transmitted through a six-fold purchase to the end of the wrench. Three of these wrenches were broken, two by tearing apart the tension side (of the rail bar) and one by an inclined shearing of the compression side. The fourth one was strengthened on both sides by the addition of a six-inch channel bar riveted to the foot of the rail.

In screwing out the arches, advantage was always taken of the temperature, by screwing out the lower members during the early and cooler part of the day, and the upper members during the warmer part; both members were kept moving out together as far as was possible. The screwing out of the outer ribs gave much more trouble than that of the inner ribs; for these received little or no compressive strain from the cables—the only one acting being the sixth, which acted on but six tubes, and even on these at an unfavorable angle. To make up for this deficiency of compression, it was necessary to aid the action of the wrench by raising these outer ribs with the tension rods on which they were supported. By alternating the action of the screw with the tightening up of the suspension rods, these ribs were finally gotten out to their correct length. Finding in the first span that the compressive strain thrown upon any tube by the action of the wrench could be relaxed by lifting the ribs with these rods—the number of the rods was much increased in the other spans when it became necessary to screw them out.

IV.—Possibility of removing and exchanging any Tube or other Piece in the finished Arches.—From the experience gained in erecting, connecting and screwing out the arches of this bridge, I feel confident, that should occasion require, it is quite possible to remove any part of the arches without the use of cables above, or false work beneath. For instance, a tube could be removed and another inserted, with no risk to any part of the structure, and but partial interference with the traffic, in the following general manner. Suspend the rib from which the tube was to be taken, upon the adjacent ribs by means of diagonal suspension rods, in a manner similar to that whereby the outer ribs were supported during their erection. The total dead weight on one rib does not exceed one ton per foot of span; this would be 520 tons for a rib of the centre span, to sustain which would not require a large amount of material. In every case, this weight could be distributed over at least two other ribs, without exceeding the moving load for which they are calculated. The weight of this rib being on these rods, the adjustable screws could be relaxed sufficiently to relieve the respective members from tension and compression. The rib could then be disconnected at any point and a new tube inserted. The temperature at the time should be considered, and all means taken to hold the rib in its position by struts and tie rods, in order that it may not move from its proper place, and thus prevent or delay the insertion of the new tube. If all material were previously prepared, the whole

operation need not take a longer time than 48 hours, and would perhaps be possible within 24 hours.*

The removal of a skew-back tube would be more laborious, but still within possible limits. These tubes being screwed into the skew-backs, it would be necessary to remove another tube first in order to obtain working room to unscrew them; it would be comparatively slight labor to remove any other part of the arches.

These cases may ever be hypothetical, but, nevertheless, they are interesting and peculiar points in the construction of this great work.

V.—Method of Painting the Interior of the Tubes after Erection.—The tubes as well at all other pieces of the bridge were painted on all sides before they left the shops of the Keystone Bridge Co. But it was also necessary that the interior of the tubes, and especially their joints, should be thoroughly painted after the tubes were in place. This could have been done for each tube as the work went on; but it would have interfered very much with the progress—especially as it was not a specified duty of the contractors. Various methods of doing this had been under discussion up to the time of the completion of the inner ribs of the first span to the central tubes. From the necessity then, of performing the work in a short time, so as not to delay the insertion of the central tubes, all plans were reduced to a consideration of which would be the most expeditious and yet effective. This induced the adoption of the following; a small spindle-shaped vessel of a size such as would pass freely up and down the tubes beneath the steel pins which go through the ends of each tube (this space being about 4

* After the completion of all the iron work on Span I, and most of the wood work of the upper roadway (March 24th–26th), two tubes in the lower members of Ribs *B* and *C* (*B*, 19–20 and *C*, 18–19, Figs. 2 and 3), which had been injured during the erection, were removed by the writer and new ones inserted instead. At this time these ribs were practically loaded with their full permanent load, and were unsupported by any cables. The delay in replacing these injured tubes was due to the length of time required to prepare new ones; but one tube was removed at a time. As previously described, the weight of the ribs operated upon was, by means of diagonal rods, taken from the adjacent ribs (Fig. 3), such additional rods as were required being borrowed from the other spans. After these rods were sufficiently tightened for a few joints each side of the injured tube, the diagonal rods running in the opposite direction, tending to hold the rib down, were also strained, thereby providing against any tendency of the arch rib to raise at this point, and bring a cross strain on the upper member. The lower adjustable screw was then gradually slacked, and the strains in the rods carefully watched, to see the tendency of the rib. It was found that slacking the lower screw ½-inch without changing the upper one, would permit the removal of the injured tube and the insertion of the new one. When this was inserted and connected, the adjustable screw was screwed out to proper length, and the strains gradually removed from the supporting rods. But 7 men were employed in all the operations. The simple operation of changing a tube did not require over 12 hours, but the work of preparing the supporting rods, cutting out and re-riveting the braces, &c., for the two tubes, took 4 days; with more men, all this for one tube could have been done in 12 hours. (Noted September 27th, 1874.)

by 5 inches), perforated at the top and sides with fine holes, its bottom filled with lead, and the whole set upon rollers, was fastened by a movable joint to a gas-pipe in 12-feet pieces which were together equal in length to that of the semi-arches. This apparatus was prepared and set to work within 24 hours—it was first forced down the whole length of the tubes, joint by joint, until it reached the skew-back, and then attached to a force pump by a flexible hose, and thin asphaltum varnish forced down the pipe. A steady spray of the paint was projected against the top and sides of the tubes, and the surplusage ran down along the bottom; the pipe was slowly withdrawn, the pump being kept at work. This sprinkler being made too weak, soon broke, and was replaced by a simple gas-pipe, closed at the end, and perforated on all sides with numerous fine holes, their aggregate area being proportioned by experiment to the capacity of the pipe, which was one inch in diameter.

This method, considered with regard to painting only, was a wasteful one; but as it was determined to fill the lower or skew-back tube temporarily with the mixture, in order to allow it to penetrate the joints of the staves, screws and backing, and protect them from the attack of water, to which they would be most exposed, should there be any leakage or condensation within the arch-tubes, the waste was nothing, for even this method required only enough material to give a surplus sufficient to fill these lower tubes, (about 1½ barrels for each skew-back tube).

The plan worked readily when the weather was warm enough to keep the varnish thin, and was so satisfactory, that the same apparatus was used in all the arches. Had preparations been previously made at the skew-backs, to draw off the surplus material, and use it over again, there need have been no waste under any circumstances.

Tests of the Bridge, when complete, (added Sept. 27th, 1874).—A 30-ton locomotive at different positions of the center span produced these results on Rib *B*.

Engine at	Deflections (Inches).		
	At Joint 11.	At Centre.	At Joint 33.
West Quarter, Joint 11	0.516	0.12	—0.06
Centre of Span	0.216	0.6	0.12
East Quarter, Joint 33	—0.024	—0.012	0.72

Similar observations were made at other points. The same load at different points produced the following results:

Engine	Deflection (Inches)—at Centre of		
	Span I.	Span II.	Span III.
At Centre of each Span.....	0.636	0.6	0 636
Moving 5 Miles per Hour.....	0.5	0.42	0.5
" 10 " " "			0.75
" 15 " " "		0.75	.....

Different loads, at different points, caused these deflections :

Load (allowing 15 Tons for each Tender).			Deflection (Inches) at Centre of			
Locomotives.	Weight in Tons.	Placed on Middle of	Rib *A*.	Rib *B*.	Rib *C*.	Rib *D*.
7	334	South Track of Span III	2.48*	1.84	1.33†	0.83
"	334	" " " " II	2.8*	2.15	1.37†	1.1
"	350	" " " " I	2.48*	1.8	1.27†	0.82
"	334	North " " } Span I	3.048	2.892	2.616	3.
"	350	South " " }				
"	334	North " " } Span II	3.48	3.44	3.89	3.92
"	350	South " " }				
10	...	" " " " II	2.37	....	....	.....

* Loaded rib. † Rib partly loaded.

The greatest vertical deflection of Span II, when one-third was loaded with locomotives was 1.43 inches and at the fourteenth joint ; when one-half was loaded, this maximum deflection was 1.67 inches and at the fourteenth panel, and when five-eighths was loaded, it was 2.25 inches and at the seventeenth panel. The greatest negative deflection or "rise" of the same span, when 12 of the 44 panels were loaded was 0.63 inch and at the twenty-ninth joint. With a load of 334 tons on north track of Span I, Rib *C* of Span II raised 0.3 inches—on south track of Span II, Rib *B*. of Span I raised 0.12 inches—on south track of Span I, Rib *B* of Span II raised 0.216 inches, and with the same load moving eastward over Span I. Rib *B* of Span II raised 0.48 inches ; in each case the measurement being at centre of the rib.

After each test the load was entirely removed and observations made for permanent set, which, however, was not at any time visible, nor was any side deflection of the outer ribs under the various loads discovered.

C.

THE EDUCATION OF CIVIL ENGINEERS.

A Paper by THOMAS C. CLARKE, C. E., Member of the Society.

READ JUNE 10TH, 1874.

Men's characters are the resultant of two factors, birth and education—education used in the broadest sense of the word, and meaning the development of all our faculties, both by instruction or the educations of the schools and by that larger education of the world which comes from experience and practice.

Some men are born with such great natural abilities that they educate themselves in spite of adverse circumstances and become leaders by the sheer force of genius. But such men form the exceptions and not the rule. Most of us are quite as much, if not more, indebted to education for success in life, than to our natural powers, and it is of the education of this class for that profession or art which we call Civil Engineering, which I have to speak.

The methods of this education used in this country and in England differ materially from those on the continent of Europe. Here we begin by a course of study at some school, college or technological institution, and complete our education by serving as assistants on some class of public works, for which we receive more or less payment. In England, the course is the same except that there boys begin younger and with less perfect theoretical education; then they serve their time under some practicing engineer from three to five years, and pay him a premium of £300 to £500 for the privilege. This they do, because their chance of future employment depends on being personally known to some engineer in large practice. On the continent of Europe, practical training by serving on public works as an apprentice to some engineer is unknown.

Education begins at the other end, by the compulsory acquirement of a high degree of theoretical knowledge. Partly with this and partly afterwards, a certain amount of practical information is given, but the leading idea is to make a man a thorough engineer theoretically before he begins or is even allowed to practice.

In this country, the tendency, of late years has been towards the continental methods. We have attempted with a wise eclecticism, to combine the advantages of both systems, and educate our engineers in the theoretical principles of the science first and then let them acquire practical knowledge by practice itself.

The important question for us to consider, and to which I wish to call your attention to day, is, whether a classical education should be combined with a purely scientific course or not, in that earlier part of engineering education which is got in the schools. We can answer this question only by considering what is the object of education at all?

It is to enable us to know ourselves and to know the laws of nature. To know ourselves, we must know the capabilities of the human mind, and this will best be found in the literatures of nations. To know the laws of nature, we must study the physical sciences. The study of letters is the history of the development of human force and will. The study of nature is the study of those forces which lie outside of human will. It is the office of the engineer to subdue these forces by human will for the benefit of man—or as the motto of the British Institution of Civil Engineering has it "Civil Engineering is the art of directing the great sources of power in nature for the use and convenience of man." Man and nature, then, should be his study. These are the views of Matthew Arnold, who strengthens his own authority by the opinions of other distinguished educators. My own experience coincides with this; observation has shown me by numerous instances, that those persons who have come to the study of Civil Engineering after a thorough training in both natural science and also, to use the good old expression, "the humanities," are those who make the most rapid progress. When a man has learned how to learn, he can quickly learn anything.

The limits of this paper will not allow me to point out in detail the course of studies which should, in my opinion, be pursued. But I may say generally—it is of the greatest importance that what Bacon calls "the relative values of knowleges" be understood, and not too much time be given to one class to the exclusion of others. The mistake of classi-

cal teachers is, that they value grammar higher than literature itself. On the other hand, the mistake of teachers of science, is that the mathematics preliminary to a knowledge of the laws of nature, is valued higher than the knowledge itself.

Much time is wasted in our colleges and technological schools over the higher mathematics. Every engineer here will agree with me that the cases where the use of the higher calculus is indispensable are so few in our practice, that its study is not worth the time expended upon it, and we have the highest authority for saying that unless its use is constantly kept up, we become too rusty to use it at all. Unless the student possesses an extraordinary genius for mathematics, I would limit its study to the ordinary analysis.

The time saved I would devote to the study of natural science in some of its branches. What is it? It consists of observations of facts and the relation of phenomena, of drawing conclusions from the data thus acquired and of finally verifying these conclusions by observation and experiment. This is the best mental discipline, for it strengthens both the observation and the judgment. It teaches patience, accuracy and a love of truth for its own sake. I see no reason why the study of the properties of all materials of construction should not be made a branch of natural science and studied, not alone by taking for granted what the books say, but by observation and actual experiment.

The *curriculum* should include the study of both ancient and modern languages; the object being to read and understand what is written in them and not to be a philologist. But let it not be forgotten that the study of our own noble language is worth that of all the rest put together. Indeed it is not sure but that the greatest advantage that we derive from the study of other languages is, that it enables us better to understand and appreciate our own. In regard to style and expression, any man who can think clearly, can write clearly. The best style is that which conveys my thoughts to your minds by the line of least resistance.

Let us now suppose that our student has finished his college course. Instead of entering the technological school, I think he should at once go into the field and the office, and learn the actual practice of his profession. His training will have given him the power of generalizing and he will be able to see the principles which underlie all classes of practice, and to distinguish what is most necessary to be learned. He will very

quickly find out the reason why things are done. After some time spent in this way in getting experience of how the affairs of the world are really carried on, not how they are said in books to be done, he may, if he can spare the time, go back to the technological school. His practice will have shown him his own weak points and the "relative values of knowledges" taught there, and he will lose no time, but apply his whole force to that which he most needs.

The practice of Civil Engineering is now so much specialized, that by this time he will probably have selected the branch that he means to follow. That branch is the one that he should study in the technological school. Then comes the time, when the value of knowing one thing well, will be appreciated.

If his abilities have obtained for him valuable employment or connections which he is unwilling to break up by going out of business and back to school, he can pursue the special branch of technical knowledge he requires, either by himself or with the aid of a private tutor. If, for example, he takes up one of the branches of hydraulic engineering, let him concentrate his mind upon the science pertaining to it. If he prefers iron bridges as his specialty, he can find occupation for all his powers in studying the application of statical laws to this class of structures.

By this means, whatever he tries to learn he will probably learn thoroughly and well. On the other hand, if he attempts to cover the whole field of applied science, he will have a smattering of many things and have nothing well.

To recapitulate—let the earlier studies tend toward developing the mind of the student, strengthening his powers of observation and judgment, and teaching him to generalize. Then let him go into the field and study actual practice, select his special branch, and acquire the technical scientific knowledge necessary to the practice of that branch. Let him learn general principles in the schools and technical knowledge while in practice.

But a few days ago the greatest of American railway managers and engineers passed away from among us. The life of John Edgar Thomson teaches us some things which it is well to consider:

First.—It teaches us first the value of *concentration*. He gave his whole mind and heart and soul to his business. As has been well said: "It was his birthright, his education, his pride, profit and amusement," and well did it repay him for his pains.

Second.—It teaches us the value of *training*. The method and discipline of his early engineering studies and practice were of the greatest value to him when conducting the larger operations of his mature life. His knowledge of the value of that training has been shown by the kind of men whom he has left as his successors.

Third.—We see that no man can hold the position that Mr. Thomson did without a very broad and general grasp of intellect. He may be a specialist and know some branch of his profession much more thoroughly than the rest; he must have much special knowledge of the details of all its branches; but all this will avail him nothing unless he has the power of generalizing, which enables him to look at his business as a whole and appreciate the relative values of all its different parts. And this power is naturally given to but few men, but in all it can be vastly increased and developed by the power of education.

I will conclude this paper by recommending the young engineer to take for his motto these noble words of Prof. Clerk Maxwell: "Accuracy in measurement—truth in statement—and justice in action; our noblest attributes as men, because they are the essential constituents of the image of Him, who in the beginning created not only the heavens and the earth, but the materials of which heaven and earth are made."

Prof. Estavan A. Fuertes.—It seems to me that the Society does not accord to our profession the rank that it deserves among those which require a lesser, and even an inferior kind of mental effort in order to be useful to mankind. This injustice may be attributed as much to defects in our technical training as to the unbalanced nature of our general education. No one will deny that, as a class, engineers are underpaid for their responsible and difficult work, and this fact must be considered as a fair standard for estimating the value that the world sets upon their services.

It may be taken for granted that any system of education that does not tend *directly* to fit the young practitioner for the successful application of the theories of the profession will be a failure. Now, then, what is the theory of engineering, or what do we understand by Civil Engineering? I cannot accept the definition quoted by Mr. Clarke. If the Civil Engineer wields the "art of directing the great sources of power in nature for the use and convenience of man," what is there left for other people to do? We only build roads and bridges, improve our coasts, excavate canals, drill tunnels, and perform other important functions with patient toil and, generally, great expense. However, I believe, that Civil

Engineering ought not to cover such a vast field of action under one title. One by one the mining, mechanical and bridge engineer has conquered each his own field, and there is a tendency, more or less recognized, towards further subdivision. Almost every engineer of note has a specialty, and study and experience are constantly leading him into untrodden fields of inquiry "in his own particular line." Hydraulics, bridging, sea-works, manufactures, canals, machinery, railroads and mining involve important functions, and almost every one of these departments is still further subdivided into specialisms. Now, I ask, can any school, in the usual time devoted to technical training, do justice to the general theory of engineering or to the study of the fundamental operations claimed, or rather included, within its domain. The answer to this comprehends the reason why the education given to what are called "engineering graduates" is so varied and heterogeneous in the different schools of the country. Our schools of engineering do not represent a concrete idea. As I once wrote to the Society* (referring to statistics enclosed at the time), it is often impossible to form any idea from the dominant studies in the curriculum, as to the nature of the courses taught in our schools. They are nearly all affected by personal idiosyncracies.

Then as to method. Some of us maintain that abstract theory will so train the mind that professional experience will, in a short time, reconcile the unsophisticated graduate to the discrepancies between theory and practice. Also, some stress is laid upon the slim opportunities for theoretical research after entering upon active business life. I would take this side of the question if there were no alternative between inapplicable theory even and the rule-of-thumb; but, since our profession demands the active exercise of a variety of accomplishments, if we can obtain a rational, practical education, we ought to strive to perfect it, and put it into operation. Also, professional enthusiasts are apt to forget that we are men, above all, and that in our contact with other men we must not narrow ourselves to being merely business or professional instruments. The great object of the engineer's education is not altogether technical. A strictly theoretic-professional education, I am convinced, engenders evil effects, restricting our capacity for the enjoyment of happiness to be derived from our intercourse, as men, with a large variety of human and charming beings who walk other paths than our own, and who can do us much good.

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* Proceedings, page 60.

Referring ourselves to the average engineer, not to the man of genius, I believe that the danger of working by the rule-of-thumb is not completely banished by the influence exerted by a strictly theoretical education, while, on the other hand, it tends to produce utopian controversalists, and men blind to the hints of experience. If they are naturally timid they will never accomplish anything, and if they are plucky they will blunder along for a much longer period than they ought.

Another class of engineers, generally the older practitioners, believe that a young man should go into the field before entering the schools. Is not this due to their individual experience with the evil effects of the purely theoretical method of teaching? It seems to me that there is an anachronism in this sort of back-acting system, which is opposed to nearly all kinds of experience.

It is not improbable that the scope of our profession is so often misunderstood, because we have not taken pains to define its limits, telling the world just what we pretend to be, and what we are capable of doing. If our attributions were more specifically known, it would be easier to shake off the numberless tradesmen, machinists, surveyors, and rule-of-thumb men, who now find shelter under the elastic or ductile name of Civil Engineer.

I also believe that if the power possessed by the Society in shaping public opinion should tend to divide our profession into definite specialties, the universities and colleges would bring about the practicable improvement of establishing particular courses for them. I see very clearly that this would at once ameliorate some of the evils resulting from our present system or rather necessity, of teaching in the abstract for the purpose of covering a wide range of "applications."

With regard to the amount of mathematics that the engineer should study, I should like to have the question settled one way or another, after careful consideration. I do not see my way clearly in this question, that is, to my mind, more transcendental than is generally regarded by most men in active practice. It is true that there are very few cases in which the engineer *needs* the higher analysis; that not one in a hundred uses it at all; that it cannot possibly be applied unless we are quite familiar with it; that, like correct fingering on a musical instrument, it makes severe demands upon our time, and that it is easily forgotten, because much of its machinery or action is dependent upon forms or processes that must be memorized for instant choice when needed. But will engineering thrive without the higher mathematics? Can we safely trust to

"hiring the services of the mathematician" to work out our problems if either we cannot understand his demonstrations or he cannot grasp the aim of our inquiry?

For the sake of discussion let me propose the following engineering specialties, and a rapid outline for a course of studies:

Specialties—Railroad engineer, bridge engineer, water-works and canal engineer, hydrographic engineer, mining engineer, dynamical engineer, geographical engineer, industrial engineer.

Requirements—Previous graduation in a classical course—a higher mathematical standard for admission than is generally required at present by the schools; two years' abstract study (with practice) on such subjects as form the common ground-work of the engineering specialties; one years' additional study in sciences tributary to engineering in detail, together with a system of lectures and recitations under different specialists, for the purpose of developing predilections among the students for any one of the engineering specialties; after the student, with the approval of the faculties, has selected the specialty best suited to his inclination and temperament he is to be handed over to the care of a "special" professor for one additional year, and this training to lead to a bachelor's degree; after two years of additional *special study and practice*, the full or advanced degree to be conferred.

This will be equivalent to six years of technical instruction, with drawing, laboratory, field and machine-shop practice, excursions, office-work, &c., besides the training in *humanities*.

That six years is not too long a time for the purpose no one will deny, and that it will not become a serious bar to the education of poor young men can be proved by the number of indigent students who have gone and are going through our present courses in this University,* and who take a post-graduate course. In fact the large majority of our students *come to us* from the ranks of the indigent grit of the land.

In conclusion, I will add what I need hardly state, that I concur fully with the writer regarding the study of language and humanizing subjects. Technical fitness, *per se*, does not insure engineering success. Besides professional eminence, the engineer must possess business capacity and tact, and he is constantly called upon to employ language demanding the most difficult kind of eloquence. He can never follow the sympathetic advice of Horace: "*Si vis me flere dolendum est primum ipsi tibi.*" The tight pockets of financiers are proof against the

* Cornell University.

emotions, and Mr. Clarke's line of least resistance is always referred in engineering to that set of coördinate axes representing the most sordid lines of human action. Since the engineer must use language elegantly and forcibly, his education should provide for this necessity.

Mr. Francis Collingwood.—In the admirable paper under discussion, it seems to me there is hardly enough stress laid upon the original meaning of the word "educate." It is a question whether *without* "natural powers" we could ever succeed at all in life. The process of educating is not alone that of instructing or adding knowledge; it has also the more important object, as the word implies, of leading, drawing out, and developing the natural powers already possessed. It is just here that most educators in our schools, high and low, make a fatal mistake. Instead of trying, by a systematic process, to bring out the analytical and synthetical faculty, to educate and control the imagination, and to develop accurate and thorough habits of observation, their chief aim seems to be to load down the *memory* with a mass of facts, often poorly arranged, and which the man has, in after life, to systematize for himself.

No one will deny that education, as a whole, is a life-long process. The writer very justly observes, that "when a man has learned *how* to learn, he can quickly learn anything." The important thing, then, is to teach the youth *how* to learn. In this view the knowledge gained at the school becomes the framework which the man is all his life-long building upon, and which he will make *complete* only in that part, which is most useful to him. The framework is essential, the filling out may or may not be complete.

Applying these general principles to the education of an engineer, to what do they lead? Certainly, first of all, to the fullest possible preparation in analytic and synthetic methods. It may be, as said in the paper, that an engineer does not need the higher calculus; but he does need the keenness of thought which results from the precise applications and broad generalizations incident to that study. The fact also, that if necessary he can make investigations for himself, which otherwise he must take second-hand, gives a feeling of confidence that can hardly be replaced by any amount of experience.

The study of natural science is an essential. This can be pursued, however (as it is in some schools), as a rest and change from the other. The sciences should be studied in their broad principles, and a thorough ground-work prepared, which can be filled out at leisure—the time

occupied need not be great. One important point here, is to be properly directed where to find correct information, then every man will study in that direction which his practice requires.

As to the languages and general literature, the writer is, no doubt, correct; the only question here will be as to what shall be the standard of admission to the technological school, where it would seem that special branches only should be followed, and everything else be considered preliminary.

Every man will be a law to himself, as to practice in field and office. There is no difficulty, however, in so arranging a course of study that the use of instruments, and that observations, measurements and notes on structures in existence, or in progress, shall make an essential part of it. The experience of our best engineering schools will show that this method is producing very excellent results.

Prof. De Volson Wood:—This paper is broad enough to open up for consideration the whole subject of education, both general and technical; preparatory and advanced. The writer makes two prominent points, namely, the character of an early education and the time of entering upon a technical one.

As to the former—for several years there has been a marked diversity of opinions among educators, especially in regard to the use or exclusion of the *dead* languages; which doubtless arose from the fact that the study of these occupied the greater part of an educational course, while the modern sciences were developing and attracting the attention of students. But whatever was the cause, the result was the establishment of courses of study from which all foreign languages were excluded; these in many instances have been modified to admit modern languages, and thereby made more literary. This caused a change in the old classical courses—so that the modern languages were also taught and natural science—thereby making them more scientific.

The old systems which have stood the tests of time are worthy of high regard; what was weak and worthless has been eliminated from them, and the useful retained. To-day generally, the best are the old systems modified instead of the new ones introduced but not established; especially if the old contain most of the elements needed in the new. It is to be regretted that these modifications cannot be made without proposing the *new* which radically differs from the *old*, but it becomes necessary because those who hold to the latter firmly oppose any change. This is illustrated in theology and politics as well as in education.

I believe that the best early education includes the study of one or more foreign languages, either *live* or *dead*—which I do not say, perhaps the latter, if not both. The remark of the writer "that too much grammar and too little literature is taught" may be true, but who shall determine the proper limits of each. Too much literature and too little grammar would make surfeited students. I object to a "sweet-meat" education—one that sickens the stomach without giving muscle to the man. If there is to be an error either way, let it be—as in structures "on the safe side"; the foundation and frame-work should be well made. But all grammar and no literature may fail to present those attractions which are desirable in order to stimulate the student to greater effort. He may study grammar from duty and pursue literature from a love of it—both should go together. The same principle applies to the study of the sciences and of mathematics.

But the broadest statement in the paper is substantially this—the man having chosen his profession and pursued a general course of instruction, should go into practice before entering upon a technical course. It may be a sufficient answer to say that this is not the course pursued in the professions of medicine, law or theology. If adopted, it would revolutionize the system which has grown up in this country, and, in my opinion, would kill all of our technical schools. The idea, in the abstract, may be good, but our ideals are rarely realized; they are modified by surrounding circumstances. If there is any science in engineering, it should be taught as a science. If one goes into practice, circumstances may prevent him from knowing much of the profession. He may be compelled to run in a narrow rut, and not even be so well prepared to choose that branch to which his tastes are best adapted, as he would be, if he had gone over the several subjects as taught in a technical school.

The plan, too, would tend to make the teaching fragmentary. One man may decide to be a bridge-builder, and his whole purpose would be to study the principles which pertain to that subject. He would care nothing for hydraulics, or road-making, or machines, or the improvement of water communication, or any other subdivision of his profession. To this I do not object after he has established himself in his specialty; but the point is, should not an Engineer take a course of instruction which will cover all of these, before he devotes himself exclusively to one of them. The evils which would result could only be avoided by shaping the general (or preparatory) courses of instruction so as to cover broadly all these subjects.

But I think that the practical difficulties would exceed the theoretical ones. If the man has entered on his profession and accomplished anything for himself, he may conclude that he can "pick up as he goes along," thus saving the time and expense of a professional education, and it is probable, his associations and engagements will be such that he will consider it impracticable to suspend or sever them in order that he may devote two or three years to study and attendance upon lectures. If "engineering is the science of construction," let it be taught as a science, after which let the student pursue his specialty; he will be able at the same time to continue study without the aid of an instructor.

I do not look for improvement in the way proposed. Having established good general and technical courses of instruction—in order that the technical part shall be adapted to professional practice, it is necessary that professorships should be filled by practical men—those who have worked in the field or shops, and know what is wanted there; and such men should have all the knowledge required to enable them to discuss the subjects of their specialties in a scientific manner.

To secure these conditions will require time. Field or shop practice without the schools will never secure it—nor the schools, without practice. At first the theorist will teach and his student may practice, after which the practitioner may become a professor. Having had a practical experience he will know what is best to omit from the theorist's course and what other matter should be added. When this is done, I would heartily approve as an addition thereto—a kind of University course, the plan proposed in the paper, whereby the specialist could be instructed in his own branch of the profession, without pursuing other branches.

ERRATA.—In "XCII. Draw-Spans and their Turn-Tables;" on page 135, in columns *B* and *C*, for "feet," read "Multiplication of Power," and for "49.50," read "495.0"; and on page 136, nineteenth line from bottom, for "×," read "+."

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CI.

THE LEVEES OF THE MISSISSIPPI RIVER.

A Paper by CALEB G. FORSHEY, C. E., Member of the Society.

PRESENTED AUGUST 25TH, 1874.*

LEVEES.—A levee is an embankment of earth raised along the bank of a river, lake or other body of water, to restrain its floods, when the surface is raised above the level of the lands to be protected.

The name is derived from the French verb *lever*, to raise; the past participle, *leve*, raised, by the addition of a letter, becomes *levee*. The expression seems first to have been used by the early French inhabitants of Louisiana, when they raised embankments along the Mississippi river, in front of New Orleans and vicinity, for protection against its annual floods. Like barriers used for the same purpose, on European and Eastern rivers, seem to have had no such name. It is not found in the cyclopædias, which have ample descriptions of the works constructed for similar purposes in the East. Most of the works for restraining floods are differently constructed, and like mill-dams, have auxiliary timbers or masonry, to strengthen them.

The Mississippi levee, proper, is a mere embankment of earth, elevated from materials found at the base. It is the cheapest and readiest possible barrier against the river's flood waters.

OUTLINE OF HISTORY OF LEVEES ON THE MISSISSIPPI RIVER.—The French colonists who settled on the banks of the river, at and about New Orleans, early in the eighteenth century, found that in the concave bends of the river, the high waters of winter and spring rose above the front banks and poured out into the lower grounds. Besides, in extraordinary seasons of floods, all the banks were submerged, and hence no portion of the river land was habitable, without protection.

* Prepared from a paper presented at the Fifth Annual Convention, May 22d, 1873.

The City of New Orleans was located on a concave bend, its centre at the nearest point to Lake Ponchartrain, where the Cathedral now stands, and the "Parade," or Place d'Armes, in front, is now called Jackson square. The high water-mark was about 3 feet above the banks, and the levee required was about 4 feet high. The first levee constructed here, a distance of 2,500 feet above and below this point, has never been materially changed about its centre. There has been neither encroachment by the river or advance of the bank, in front of the old city, laid out by Dumont de Latour, 5,000 feet front on the levee street, and nearly square. The survey was made in 1717, and a bastion line of rampart, with stockade and ditch, passed around the sides and rear, beyond the present Rampart street, which was the rear line.

The levee built was, in fact, a rampart and circumvallation, 6 feet high in the rear, and 4 to 6 feet on each side. The front is stated to have had 18 feet crown on its completion in 1726, nine years after it was begun. This is the oldest embankment known by the name of levee in the State. Both sides of this rampart, and the rear line, have been graded down for streets, and the front, with the occasional additions needed to replace natural wear and tear, retains its original height probably; it has been merged into the wharf and sloped into the street.

In the year 1732, we learn from Le Sage du Pratz, that the plantations extended for 30 miles above the city, and to the English Turn, 12 miles below; in front of which the proprietors were compelled to build levees. Both these termini reach high grounds, very rarely overflowed, and by raising small guard levees, running back from the front, the waters discharging over the banks on each side were guided back to the swamp, and thus kept from the cultivated lands.

The Bayou Bienvenu, 12 miles below the city, was an outlet of the Mississippi river into Lake Borgne, and the high grounds on its upper bank formed the grand levee. This little bayou, being cleared out, is now the channel for the Lake Borgne Canal, on the Repose plantation of Mr. W. C. Ervin. The present plantation dwelling, by tradition has an age of 110 years, and the levee is probably the second built, in the 150 years of levee history. Its height is 3.5 feet, say one foot above the highest known water in the river, that of 1871.

The vicinity of the Red Church, a very old structure, 25 miles above New Orleans, was the upper limit of the levees described, where the bank has never changed, and the levee is only 3 feet high. The guard levee here was but the earth thrown from a ditch, on the north side of the plantation. The extravasated water at this point reaches the level of the

swamp of Lake Ponchartrain, in two miles from the front. Considerable embankments in the rear of these plantations, with flood-gates and draining machines, are necessary to restrain the lake waters, which in south-east storms fill the swamp in the rear. It is highly probable that the levee here has never been raised higher than those built on the same spot by the first inhabitants, 145 years ago.*

The floods of this river, in a state of nature, kept the banks well up to high water-mark, except in caving banks, or concave bends, where the height of banks would fall 2 or 3 feet lower. This was my observation of the habitudes of the river, above Red river, prior to the building of levees. After the country was protected from overflowing waters, it is manifest that the caving banks soon drove the levees back to a point so low, that higher levees were required to defend against the same height of floods. Hence, the common impression that the river's bed is filling, and the high water floods more elevated than formerly. Many banks are to-day nearly as high as the highest water-marks.

The work of reclamation proceeded very slowly. The task was a great one, the inhabitants few, and the crops produced not very lucrative. Neither cotton nor cane was then extensively grown; but indigo, Indian corn and rice formed the staples. So in the year 1752, according to Monette and to Martin, as also to Pittman in 1770, the extent of settlements was limited nearly as above; but both sides of the river were leveed to some 30 miles above, and only 12 or 15 miles below—probably 20 miles on the right bank, down to the old Fort St. Leon, now the plantation of E. F. Villere, Esq.†

During the last half of the eighteenth century, the provinces languished. Still fragmentary levees were built, say 25 miles front in Pointe Coupée, and 25 miles on each side down the Bayou Lafourche. The colonies passed into Spanish hands in 1766, and were acquired by the United States in 1804. A new and powerful stimulus was given to the work of reclamation. In 1812, the most of the right bank, from Pointe Coupée, was leveed, down to the lowest settlement; except where the country was unoccupied. On the left bank the levees were, for most part, continuous from Baton Rouge down to about Pointe à la Hâche, 40 miles below New Orleans.

On the old river bed, called Fausse Riviere, in Pointe Coupée, the land had been so relieved from overflow, by the cut-off made in 1722, as to render it habitable with very small levees. It was leveed prior to

* For 30 years the writer has known it unchanged, with 50 years tradition before.
† Gayaré Lectures.

1812, in the portions requiring it. By the year 1828, the country below Red river was all leveed on both sides, except the left bank above Baton Rouge, where the river runs along near the bluffs.

The flood of 1828, exceeded all previous, and discouraged levee building. Yet between that year and 1844, levees were rapidly applied to the right bank above Red river, and completed mainly to the mouth of the Arkansas; however, below Vicksburg, the Diamond Island Bend, for some 25 miles was not continuously or generally leveed. The levees left the river bank at New Carthage, and were built along the Bayous Vidal and Roundaway; so that the high waters of this bend were allowed to collect in the Roundaway and Willow bayous, and thence discharge into the Tensas.

At that period, however, the whole alluvium, between the Mississippi river and the Ouachita hills, was nearly redeemed by the levees on the Mississippi, and inhabitants swarmed upon those lands. They clamored for the completion of the levees below Vicksburg, because the floods of 1844–49–50–51, drove them from that quarter; they were answered by being included in the levee districts and taxed for levees.

Between the years 1850 and 1860, the levees of the right bank, from Cape Girardeau down to near the mouth of the Arkansas were built piecemeal, but finally were nearly continuous, leaving intervals of less than 40 miles, when the war of 1861 ended all improvements.

The State of Mississippi enacted laws for the building of levees on the left bank, from her northern boundary, Lat. 35°, crossing the Yazoo Pass, 12 miles below the city of Memphis, and extending down to near the mouth of the Yazoo river, say 10 miles above Vicksburg.

These levees were built by an *ad valorem* tax upon the alluvial counties, and were completed before the war. However, the levees were mainly built by the owners of front lands, and grew by degrees for a number of years, beginning about 1840, and continuing to 1850–51, without other aid than private enterprise. The counties then were charged each with its own levees, until about 1857, when, as we are informed, the whole Yazoo basin was made a single levee district, and the levees were built by commissioners, with funds raised from the entire district. The completion of this system reclaimed an area of over 6,000 square miles of the best cotton lands known to husbandry.

Form and Dimensions of Levees.—We have no definite information as to the form and dimensions of the early levees, erected for more than 100 years upon the banks of the Mississippi river. The first levee thrown up around the city of New Orleans, we are told, had a crown of 18 feet,

but this was more for a rampart, public landing and walk, than for mere defense against the waters. Such a crown, with slopes of two to one and 4 feet in height (the probable average), would give a base of 34 feet, and a cross section of 104 feet, a quantity entirely needless for a maximum pressure of 3 feet of water, or some 6 feet at the rear of the rampart.

Very many of the old Creole levees still exist, with crown of 4 feet or less, and slopes of two to one on both sides. A levee 4 feet high, with these dimensions, would have a section of only 48 feet, and this would be entirely safe, even if built of the least tenacious materials. Such levees are now found, with the live oak and pecan trees, or other oaks, planted upon them, having an age of 75 to 100 years. These levees have never been renewed, though occasionally repaired, to compensate for abrasions by being used as a foot-path by man and beast. It is probable that this was a common form of levee for at least 100 years. In concave or caving banks, of course, the dimensions were in proportion to the needs, but the levees were mainly of this form. Still a few on high grounds were made (probably by opulent proprietors) mere elevated public roads, of dimensions ample for carriages to pass each other on the crown. Some of these still have the lines of shade trees on the slopes which pass gently into the natural earth.

As a matter of physics, and pertinent to levee strength, it should be added, that all levees of long standing, have their front battures elevated by repeated overflows, up to near the great high water-mark. This adds greatly to their security, and leaves little for the levees to resist in the way of mere pressure. All the old levees are covered with a turf of Bermuda grass, which protects their slopes from abrasion by weather or occasional travel, and this is now universally required.

The Louisiana levees of the early period appear to have had such dimensions as the proprietors chose to adopt, with some local authority to compel adequate strength and height. In the front of improved lands the stimulus of interest was generally sufficient to secure an adequate levee. Under the land grants of the French Government, the construction of a front levee was a servitude imposed upon the proprietor, and a condition of title: this was generally enforced by the police authorities of the parishes along the rivers. In fact, the United States surveys conformed to the French system, and laid out the lands fronting the navigable streams in the French form, with a depth of 40 arpents, and continued the servitude as to levee building for many years after the cession of the territory. But no form or special strength was prescribed by the United States. The police juries (County Commissioners) enforced the

servitude. The larger levees, such as that of Fausse Riviere, at Waterloo, Bayou Cut-off, in Lower Concordia; Bullitt's bayou, above Vidalia; L'argent, below Waterproof; Carthage, at Bayou Vidal; and Providence, at the mouth of that lake, were built under engineering direction and advisement; all others were measured and directed by the parish surveyor, generally not an engineer, hence the dimensions prescribed were various, but generally sufficiently strong for the servitudes.

We come, in the progress of experience to a period of extensive interior settlement on lands reclaimed by the labors and expenditures of the front proprietors, and at the same time to a period in which the servitude was exhausted. Riparian owners plead their having built one levee, many of them two or more, in retreat from the caving banks, and they appealed to equity and sustained the claim, that all, in proportion to values, should contribute to the maintenance of levees, since all cultivating alluvial lands were alike protected by levees.

After the year 1855, this general doctrine was adopted above Red river, and within the next five years, the same doctrine was acquiesced in throughout Louisiana and Mississippi. Commissioners were elected by the owners of alluvial lands, for each levee district or parish—Carroll and Madison formed a district, and the parishes below, down to Pointe Coupée, inclusive, were made levee districts. These Commissioners had ample power to prescribe new levees and repairs, to levy proper taxes for their completion, and to call out the slaves for special works, to let contracts, and generally had jurisdiction over all levees. The States of Arkansas and Missouri, in like manner, established levee systems, more or less recognizing the equity of a general tax upon all alluvial proprietors. The fund derived from the swamp lands ceded to the several States by Congress, to aid in their reclamation, was used for this purpose.

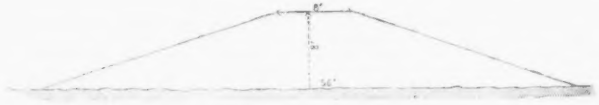
With such views prevailing, the system of local and personal responsibility was greatly modified, and whole communities became suffragans, in levee voting. The commissioners employed engineers, enacted rules for levee dimensions, and raised this work of protection to the dignity of a profession.

The earliest enactment I have been able to find respecting levees, relates to those above Red river, in the old Parish of Concordia, extending from the mouth of that tributary, Lat. 31 , to the upper limits of the State, Lat. 33 , a river distance of 300 miles. That law required a levee of such form that there would be an increase of base downward from the crown—thus, the crown to be at least 4 feet wide, the slope of base not less

than two to one for 3 feet perpendicular depth, and thence to the ground, for each foot in depth, an increase of base on each side equal to the distance of its upper face from the crown.

Concordia Levee, ordained about 1835.

About 1854–5, the State of Arkansas, established by enactment, certain dimensions and restrictions relative to levee building, which was to be at the cost of the State. These laws provided that all vegetable matter should be excluded from the body of the levees,* the height was fixed at 30 inches above high water-mark, the breadth at the crown made equal to the height, and seven times as great at the base, and all public roads crossing the levees to have a slope of seven to one.



Arkansas Levee of 1854-5.

The State of Mississippi, by law prescribed that all levees shall have a crown 3 feet wide, and a base in width equal to six times the height; a levee 8 feet high therefore had a base of 48 feet.

In 1850, and for two years thereafter, the Senate of Louisiana instructed its Committee on Rivers and Levees to examine and report upon a system of levees for the State. An elaborate investigation was made, assistance was generally asked from engineers and others in the State familiar with the subject, and the writer was employed to make surveys, examine the physics of the river and to devise and report a plan. The report was printed in 1853: it embraced maps of the alluvial lands of the Mississippi river divided into five districts or basins, and many sub-districts or local basins were suggested and pointed out. It contained a levee formula, the adoption of which was urged as a part of the system prepared and commended whereby levees were to be constructed under State control and by a State Engineer with assistants in each district, the whole cost to be a tax on the alluvial lands.

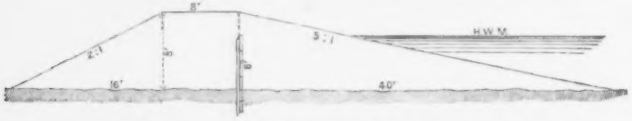
The levee formula was thus stated: H = height, C = width of crown,

* This exclusion of vegetable matter has been and continues to be evaded all over the levee region of the delta.

B = that of base, and S = area of section, then relations as follows were established; $C = H$ and $B = H^2$, whence

$$S = \frac{H^2 + H^3}{2}.$$

The land slope was taken at two to one, and the river slope at five to one. "A close fitting horizontal cypress board fence, secured to posts," was "to be planted 2 feet in the ground, be buried in the levee under the front edge of the crown," and rise above high water-mark.*



Forshey's Levee of 1850.

The levee was not put in use, nor was the State system adopted which was recommended. The dimensions ultimately fixed by enactment, in 1855, but never very thoroughly put into use, was as follows: every levee having 3 feet of water against it, to have a base of five times the height of the water, and for each additional foot of water pressure, the base to be extended by the multiple of the distance from the crown, say 2 feet above the water surface. This was nearly the same as the Concordia ordinance, and the formula was a favorite one of the earlier French levee builders.

After the war, state bonds, payable forty years from date, were issued and sold to rebuild the levees, whereby the burden fell upon the whole people of the state. A Board of Levee Commissioners was created in 1859, but did not act until 1865. Engineers were employed who entered upon the construction of levees, each seemingly after his own method and form. Many of the large levees constructed in the ensuing six years, have slopes of two and five, and some three and three to one; occasionally the long slope was built on the land side of the levee. Within this period the state constructed about 14,124,371 cubic yards of levee, at a cost of $8,245,000, or nearly 60 cents per cubic yard.

After three years of most expensive work, the State of Louisiana chartered a company, and contracted with it for the construction and maintenance of the levees for 21 years, engaging to pay per annum ten per cent. of the cost, computed at 60 cents per cubic yard, the whole construction being limited to 15,000,000 cubic yards, and to be built within four years from 1871. This contract was modified so that payments are

* See Senator Ricker's Report published in 1853.

to be in cash and at a rate of 50 cents per cubic yard; the amount and location to be prescribed annually by a Commission of Engineers chosen respectively by the Governor of the State, the Levee Company and the President of the United States; the commission to determine the form and dimensions of all new levees, and locate them on the ground—from their award there is no appeal. In addition the Company is to keep up or repair levees, for a fixed amount of money, realized from a two mill tax on all the property of the State. These amounts will not probably fall below $1,000,000 annually, and may reach $1,200,000. The work done under the contract, before modification, in the two years of the company's life, was 3,630,000 cubic yards.

The length of levees on the Mississippi river, existing and required, may be summed up as follows:

		Miles.	
Louisiana.	Left Bank—Fort St. Philip to Baton Rouge	205	
	Right " Fort Jackson to Lafourche Bayou	135	
	Donaldsonville to Red River	157	
	Red River to Arkansas Line	278	
			775
Arkansas.	Mouth of the Arkansas River	109	
	Up the Arkansas River to Pine Bluff	40	
	White River to Helena	96	
	State Line near Hickman	300	
			545
Missouri.	State Line to Cairo, 43; Commerce, 30	73	
	Cape Girardeau	4	
			77
Mississippi.	Left Bank, Vicksburg to Memphis, less 20 miles		380
	Total		1,777

The length built by Louisiana should be increased by:

Red River	200	
Black River and Tensas	200	
Atchafalaya	90	
De Glaize	40	
Lafourche	150	
Plaquemines	20	
Fourdoche and Grosse Tete	20	
Old River	20	
Rouge and Boeuf	40	
Altogether making the Mississippi front		780
To which add the amount above		775

And Old River Lakes as follows:

Concordia	35	
St. John	20	
Bruin	15	
St. Joseph	30	
Providence	30	
		130
Whence total length of levees in Louisiana		1,685

If the levees that have been rebuilt—500 miles of the very largest class in the concave bends—are added, the cubic contents would be swelled by half. At a moderate estimate, the first amounts to 50,000,000 and the other to 25,000,000 cubic yards, making the total constructed by Louisiana alone, the enormous sum of 75,000,000 cubic yards, equivalent to twenty-five pyramids like that of Ghizeh.

The dimensions adopted by the Commission of Levee Engineers, October 1st, 1871, and now in use, are as follows: all levees below New Orleans shall be 2 feet, those between the city and Baton Rouge 2½ feet, those from Baton Rouge to Red river and above the Red river shall be 3 feet above high water-mark: the details of the crown, slopes and construction, will be prescribed for each separate levee on the ground.* All levees built without a covered fence, or other impervious septum, shall have a crown in width compared with height, as follows: up to 6 feet high a crown 4 feet wide, for 8 feet high a crown 6 feet wide, for 10 feet high a crown 8 feet wide, and those over 10 feet high a crown 10 feet wide. The slopes on each side to have 3 feet base for one foot in height.

What is required.—The Mississippi river flows throughout the delta, between alluvial banks of its own deposit. On the right bank, from Cape Girardeau, in Missouri, to the Gulf of Mexico, the river nowhere leaves the uplands: its rise and fall is so great, and its deep channel so well understood, as to justify the belief that, except in a few localities, the bed is alluvial. So new, geologically speaking, is the delta formation, that its plane of declivity towards the sea, has nowhere been materially depressed or elevated by disturbing forces beneath.† Therefore the river's high water surface is nearly a plane. The whole elevation in the 1,080 miles, from Cairo to the head of the passes above the mouth, is 322 feet; or the inclination of the high water plane is 3.5 inches per mile.

The deposits of its sedimentary matter, along the front of the river's channel, where the caving is not perceptible, or is very slow, have raised the banks to near the high water-mark. This renders the height of the barrier required to control the waters very moderate along the less friable banks—that is all the convex banks, and many of those that are concave.

* Note—In consequence of the reduction of high water-mark by the "cut-offs" above Red river, the levees will not, for the present, be built to grade; but shall be 3 feet above the new high water-mark, as observed.

† The Madrid earthquake of 1811-12, effected a local disturbance, west of and parallel to the river, depressing and forming a lake of 80 miles length, 10 to 30 width, and 5 to 20 feet deep. Old forests of cypress still stand erect in the lake, but most other trees have disappeared. The plane of the river's discharge was not permanently disturbed.

In a state of nature an average of 3 feet is probably a fair estimate of elevation of the water-mark above the banks; in many places this elevation is only 5 to 10 inches; and hence, on all such banks, a levee of 4 feet would be sufficient security against the mere quiet lapse of the high water currents. For safety, to cover contingencies, the inhabitants built levees about one foot above the highest known water-mark. Engineers, estimating for the ravages of waves, and the attacks of drift-wood, prescribe from 2 to 3 feet surplus. On the concave banks, the average high water-mark may be placed at 4 feet, and the ordinary levee at 5 feet; while a safe levee should have a height of 7 or 8 feet.

But height is only one element of a well-proportioned levee; base and crown are two others equally important, and quality of material is a fourth. It is necessary to resist:—*First:* The weight of water multiplied by its depth, for each foot pressing the levee. *Second:* The insinuating property of water, as shown in passing between the particles of the material used in building the levee; this is called "*seepage.*" *Third:* The impact of wind waves, an indefinite force. *Fourth:* The impact of steamboat waves, a definite effect not yet measured. *Fifth:* The occasional lodgment of logs of drift-wood, in such position as to be oscillated by waves, and thus to wear through the levees. *Sixth:* The ravages of earth-borers and burrowers. *Seventh:* The attacks of vicious or malignant persons, so far as practicable.

These are the leading items of the service of a levee, which, with all, must cost so moderate a sum as to be afforded by those who need them. The extent of levees, in proportion to the lateral distance protected (in the unfinished condition of the reclamation), is so very great, that they must be cheaply built, if at all. In truth, in many river bends, new levees have already been thrown up on caving banks, back to the low swamps, and the structures now required are of such magnitude as to render the cost an intolerable burden to the districts requiring them.*

* A single example will illustrate how impossible it will be for the people of these States longer to defray the expense of levees. The Louisiana Levee Co. recently built, in the parish of Tensas, 320 miles above New Orleans, the Kempe levee, 4 miles long, of 18 feet average height (22.5 feet maximum), and with a base of 130 feet; 640,000 cubic yards were paid for at a cost to the State of $384,000.

Greatest cross-section of Kempe Levee of 1873.

The Kempe plantation has, in 27 years, worn back more than a mile, so that this new levee occupies the lowest cypress swamps. (See Map).

What, then, is the formula for the cheapest levee, compatible with safety? No earth levee is safe against the attacks of waves, driftwood, or malicious persons. But, considering these risks, and those from the frequent crevasses which destroy property worth vastly more than the $40,000,000 the levees cost, I answer as follows:

For a pure earth levee of *sand and clay or loam*, a crown equal to height (up to 10 feet), slopes three to one on both sides; and for one of *joint or blue clay*, a crown of 6 feet, maximum (for small levees, a crown from 2 to 4 feet), slopes two to one on both sides.

In the first of these levees, the excess of weight and tenacity over water pressure, added to current impact and wave blows, appears very great; but a regard for contingencies, as above described, causes this excess in practice. To illustrate, take a common levee 8 feet high, it has a cross-section of 256 square feet. Against the weight of this 32,000 pounds of earth (at 125 pounds per cubic foot), we have an aggregate water pressure from bottom to surface due to 6 feet head, equivalent to that from a 20 feet column, or $62.5 \times 20 = 1{,}250$ pounds pressure of water, against the 32,000 pounds of earth; a ratio of over 25 : 1.

An 8-foot levee of clay has a section of 160 feet, and weighs per foot in length, 20,000 pounds. But I assume (and can demonstrate) that the tenacity of this clay used on a levee will give a co-efficient of two (or even four), and hence $20{,}000 \times 2 = 40{,}000$ pounds resistance, which is opposed by 1,250 pounds pressure; a ratio of over 32 : 1.

Now let us allow for the impact of waves, say one foot high, running at 15 feet per second—a steamboat rate. This, according to Hewson, gives a multiple of three into the water striking the levee, which, for a wave one foot high, is equivalent to 3 feet added to the 20 feet, or about one-seventh to the servitude of the levee, whence the ratio becomes 28 : 1. Now, wave-blows occur only one at a time, their impacts are entirely separate and cannot be cumulated. True, their abrasive force, which is another element entirely, is very injurious; this should be guarded against by other means, however, than increase in cross-section.

All this reasoning brings me to the main point, namely, the suggestion of a cheap, simple and entirely efficient remedy against borers, burrowers, wave-blows, malicious injuries, drift-wood, and more than all, against the necessity of providing a twenty-fold resistance against attacks upon our levees. This was prescribed in the levee submitted by me, in Ricker's Senate Report of 1852 (see page 8), and consists of a septum or partition, impervious to water, inserted longitudinally, covered in the levee, and descending well into the earth beneath the

base.* The best and most economical is a cypress board fence close-fitting, the boards lying longitudinally and lapping on the edges in weather-board form, to be securely spiked to posts set 3 feet in the ground, the ditch in lieu of the ordinary muck ditch to be cut as low, the fence started from its bottom, the posts, say 3 by 6 inches, placed 10 feet apart and the boards secured at each lap edge by wrought-iron nails clinched midway between the posts. The lumber in a 10-foot levee amounts to only 12 feet per running foot, and costs about 50 cents.

Forshey's Levee.

This fence thus placed in the body of the levee, is equivalent in strength to earth 3 feet in width, or 1⅓ cubic yards, and a levee including it would cost little more than one of earth with 3 feet increase of crown.

But with this impervious partition, there is no possible danger from burrowers or borers, which can neither penetrate nor flank it. Even the muskrat or the beaver cannot gnaw a plane surface. Should waves or currents wear into the levee to the distance of the front of the crown, they will be met by this best of all revetments, which they attack in vain. If a drift log is so lodged as to vibrate and wear away the levee front, it reaches this barrier, and suspends its encroachments.

Whenever the material of the levee is of a character to permit seapage or permeation, the water finds its way only to the partition, leaving the body of the levee beyond the fence in a condition of dampness, favorable to maximum tenacity. But if from imperfect construction a very little water penetrates the barrier, it is taken up near the point of leakage, absorbed by the neighboring mass, and dissipated in evaporation from the surface of the levee.

Should a malicious person attempt to cut the levee, this continuous partition will deprive him of the aid of the abrading water, and before he can let any escape, he probably will be detected.

* From stubborn prejudice, this plan slept twenty years untried, while throughout the area of civilization thousands of novelties in engineering were devised, adopted and canonized. In October, 1871, the Commission of Levee Engineers adopted it for large levees, and it was used for some miles. The specifications in the contract required the horizontal boards to be added as the embankment ascends, one at a time, so as not to interfere with the runs of the workmen, the earth to be well rammed in the ditch and up both sides of the fence to the top, and the fence was located beneath the front edge of the crown of the levee. These levees have never leaked, even when pressed by 12 to 15 feet of water. Under always damp ground, cypress, heart-pine, mulberry, cedar, and most good timber, lasts indefinitely.

Whenever (and this occasionally happens) a crevasse occurs from the caving of the river banks, carrying away a portion of the levee, fence and all, the two ends left standing afford complete protection against rapid abrasion, and the work of closing the crevasse is limited nearly to the opening first made.

Thus all but the first and last of the eight servitudes of levees before enumerated are effectually disposed of; what relation this method has to the last and first of these requirements—the lateral pressure of the water, and the economies of construction—is still to be considered.

As to mere pressure, it has been shown that the sand and loam levee of the Louisiana Commission of Levee Engineers now in use, has a resistance to lateral pressure of more than 25 : 1, or if the impact of waves is added, about 22 : 1. The same levee with land slope reduced to two to one, as fixed where the fence is used, has still an earth resistance of 20 : 1, independent of the fence; the clay levee, with its double strength, due to tenacity (retaining the same formula for all kinds of material), resists in a ratio of nearly 50 : 1, and when the fence is used, after all reductions allowed by the Commissioners for wave-blows and land slope has been made, this ratio is still 45 : 1.

This method of construction, therefore, satisfies the conditions for which the enormous resistance was provided. Shall not, then, a levee be constructed under the same idea as a bridge, or an arch, with special reference to its uses and servitudes, and an ample "co-efficient of safety"? Let ten be assumed for this co-efficient, and the form and dimensions of levee be determined in accordance therewith; adopt a land slope of 45°—the angle of repose, give the levee a crown of 4 feet, and if 8 feet high, the cross-section will be 96 feet, and the resisting weight 12,000 pounds, whence disregarding tenacity, the ratio of resistance to pressure will be nearly 10 : 1.

The slope of repose, under water, of earth is about 60°; hence, with this slope a crown of more than 2 feet wide is unnecessary, since at the high water-line, 2 feet below the crown, the levee thickness would be 8 feet. Adopting this width, a levee with fence, may be constructed having a co-efficient of safety of about twelve in the weight of earth alone, the land and water slope being limited to the angle of repose for clay or other material used. For additional security in important localities, large levees and permeable material, the board fence may be doubled and built edge to edge with broken joints; since thereby the cross-section of earth may be reduced from 256 to 112 feet, the additional expense will be small, and the security against seapage effectual.

To prevent abrasion on the front of levees long exposed to the waves of high water such as those below New Orleans and in many localities above, requires a special provision. A double plank revetment, of good material, pressed well into the ground in front of, and forming the outside slope of the levee, with slope one and one-half to one, has been used with excellent effect. Wherever the buried plank fence is not required, this protection, if the levee is much exposed to waves and driftwood, may be used.

Forshey's Levee, (double board fence).

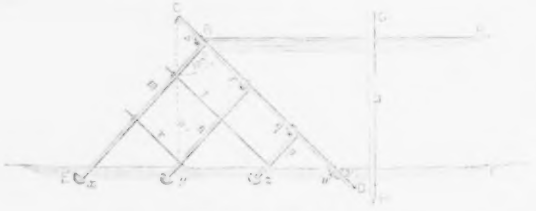
The encroachments of the river, along caving banks render it necessary in many cases to retire inland 1,500, even 3,000 feet, to sufficiently protect during their life (usually twenty-five years) the very large embankments thrown up. This often excludes a whole plantation, leaving exposed the very property to be protected, and the owner has but little courage or interest in sustaining an improvement which thus may ruin his home or plantation. Therefore, if practicable, a levee should be devised that may be retired before the advancing river and erected in a new location, thus saving the great cost of new earthen embankments, and protecting instead of destroying the best portion of the plantation—that nearest the river edge.

The large expense of a portable levee is the chief obstacle to its adoption. One may be constructed of iron as sketched and estimated on following page, 15 feet high for $31.80, and 10 feet high for $20.50—per running foot.

The Kempe levee, if constructed on this plan, would have cost per running foot twice as much as it did, but the length could have been reduced from 20,000 to 14,000 feet, and the annual expense of wear and tear and expense of removal, would have been but 20 per cent. of the other; excepting the timber, the material would have been good for at least fifty years, when another removal might be demanded, and by it 200 acres of tillable land would have been saved. In this instance (as subsequently proved), removal would not have been necessary; for the river has reacted and ceased its ravages in all the upper portion of this bend.

Moreover, in many cases, as at Ashton, near the upper boundary of

Louisiana, the floods of a few years build up by deposit the lands thrown out, and almost entirely reclaim them. In such places a small earthen levee, constructed on the river front, is sufficient, hence if, instead, a portable levee were used, it could afterward be removed to some other locality demanding it.



Forshey's Portable Levee.

Plates (*C D*), 18 feet	324 pounds.		
Beams (*q r s*), 6 × 6, 5 × 5, and 4 × 4 inches	45 "		
Braces (*m*), 4 × 4 inches, 18 feet long, (*n*) 5 × 5 inches, 12 feet long, and (*p*) 6 × 6 inches, 7 feet long	303 "		
Ties (*t*), 4 inches × 11 feet, and (*v*), 4 inches × 7 feet	108 "		
Weight of iron	780 "	at 4 c.	$31 20
Timbers (*w, x, y, z*), 16 feet; piles and caps (*G H*), 21 feet = 37 feet, worth			60
Cost of portable levee 15 feet high			$31 80

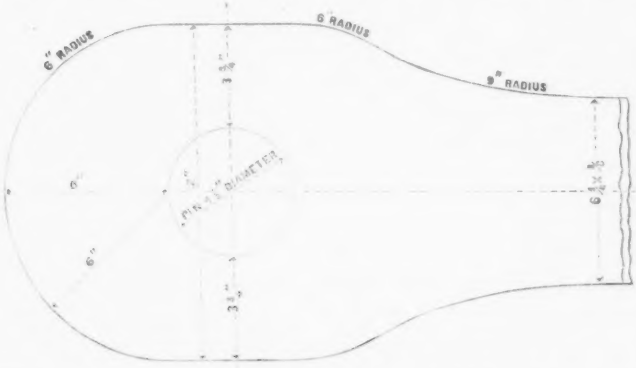
The portable levee is commended with much earnestness, for all places on the river demanding a high barrier against the floods. All levees of 10 feet water mark should be of this kind, if on a caving bank; and those of 5 to 10 feet water-mark should have the impervious board partition. These two improvements, with the reduction in section thus made practicable, will lighten the levee burden by one-half in total cost, and afford a security against floods and crevasses never yet enjoyed.

When it is considered that the area of land in the Mississippi delta, which levees only can protect, is about 38,700 square miles, not more than one-tenth of which is inhabited and cultivated (chiefly because of inadequate levee protection), while more than four-fifths of the delta is habitable and of unexampled fertility, the motive to reclaim it from the floods for the uses of man should prompt the ambition of the engineer, the philanthropist and the statesman.

DISCUSSIONS.

ON THE PROPORTIONS OF THE HEADS OF EYE-BARS.*

Mr. Charles Douglas Fox (of London, England).†—I have tried



several forms of links for India, and herewith present a sketch for the information of members, showing that form of head and size of pin which I have found to give the best result, and which I am now largely adopting.

ON UPRIGHT ARCHED BRIDGES.‡

Mr. Squire Whipple.—Permit me to submit the following comparative estimate of the amount of iron in the trapezoidal truss, and in the upright arch bridge, reduced to like spans and conditions.

The bridge over the Erie Canal on the Rensselaer & Saratoga R. R., is 149 feet long, and believed to be the first iron trussed railroad bridge of so long a span built in America. It is a single track bridge, proportioned to carry 2 000 pounds live load to the lineal foot, with stress of 10 000 pounds to the sectional inch of metal, and contains 75 000 pounds of iron ($\frac{1}{7}$ of which is cast-iron), in trusses and lateral stay system. Adding 25 per cent. to bring the live load up to 2 500 pounds to the foot, and dead load proportionally, makes in round figures, 94 000 pounds. Adding

* Continued from page 338, Vol. II. † By note Oct. 17th, 1874. ‡ Continued from page 238.

curved, &c., &c., of most of which several combinations, examples might be adduced.

ON THE EDUCATION OF CIVIL ENGINEERS.

Mr. THOMAS C. CLARKE.—All are agreed that Civil Engineering being a science and an art, must be learned both from books and by practice. Authorities differ as to the length of time to be spent in the schools, and whether practice should precede or follow a course of technical instruction. No one will deny that the true method would be to combine both theory and practice at the same time. This is done to a certain extent by students of mechanical engineering at the technical schools at Hoboken, Worcester, and Ithaca.

But can any one tell how this system can be applied to Civil Engineering? Colleges have no railroads, canals, nor waterworks to construct, and even no actual surveys to make. The construction of public works is generally carried on at places remote from the schools. The student is forced either to go first to the technological school and then into practice; or to reverse the process, as suggested by me. Prof. Wood considers it a sufficient answer to say that this is not the course pursued in the professions of medicine, law and theology. But he forgets that in the studies of medicine and law, practice and theory go hand in hand. A medical student attends the clinique, performs actual dissections, and hears lectures. A law student works in an office, attends the courts and reads law books, all during the same years. How can students of Civil Engineering do this? Only by private studies during practice. But this is too great a task for the energies of most men. After a hard day's work in the field, he must be an extraordinary man who could keep awake over his Rankine.

So I am driven back to my first position. First, get a good general education; learn to observe and to reason correctly, and to be accurate and thorough in whatever you do. Then spend several years in practice. Learn the routine of office and field work. Learn to deal with men. Then if your mind is still hungry for knowledge, go to the technological school and study the branch you mean to follow, and study it exhaustively. Find out all that has been done, thought and said about it, that is recorded in books or can be obtained from actual inspection. It seems to me that this is wiser than trying to cover the whole vast field of Civil Engineering, which usually results merely in acquiring a superficial knowledge of everything, and an accurate knowledge of nothing.

I will close by relating an anecdote, to illustrate my meaning. Two engineers were sent to make surveys and report upon the improvement of an important river. The one covered the whole extent of his territory with primary and subsidiary triangulation. This took so much time, that he could only sketch in the topographical and hydrographical features hastily and without actual measurement. He found when he got back to his office that his surveys were too general to be of any use. The other simply ran a traverse line along the bank of the river as a base, and at such points as he knew that artificial improvement would be required, he ran lines a few feet apart and took levels and soundings at their intersections. When he got to his office he had all the data necessary for an accurate estimate, and obtained by no greater labor than the first man had used. Clearly, the second man took the correct course. He understood the Baconian axiom of the "relative values of knowledges." He studied what would be of use to him.

It is because I think that students of Civil Engineering waste much time in studying things that can be of little use to them as Engineers, that I recommend a preliminary course of actual practice to form their minds in some measure, and to direct them how to find out what studies they really should learn.

NOTE ON THE LEVEES OF THE MISSISSIPPI RIVER.

The great flood of 1874 in the Mississippi River, brought the high water-mark at this point, on the Repose plantation, 12 miles below New Orleans, 0.7 foot higher than that of 1871. Portions of this levee then were at the high water-mark, and were raised slightly to prevent waves, &c., from cutting it.

It is now obvious that the two cut-offs of 1867, at Palmyra and Terrapin, Lat. 32° and 32°.30', abridging the river's length by 27 and 13 miles, respectfully, have congested the waters in the channel of the lower river. The flood level was reduced by 6 feet at Vicksburg midway between these cut-offs; and this depression was felt, gradually thining out for 200 miles below Red River; still, below Baton Rouge, say 250 miles above the mouth, and thence to the Passes, the river was higher than ever before, by a foot at Donaldson, and less at the English turn.

ERRATA.—On page 268, thirteenth line from bottom, for "Bienvenu" read "Duprez."

CII.

EFFICIENCY OF FURNACES BURNING WET FUEL.

AS DETERMINED BY EXPERIMENTS ON A LARGE SCALE.

A Paper by Professor R. H. Thurston, Member of the Society.

Read October 21st, 1874.

PART I.

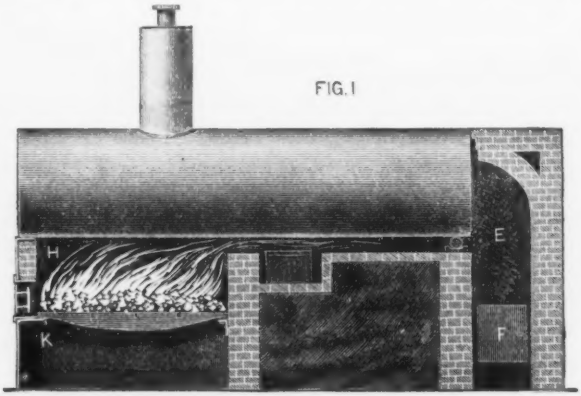
1. The writer was recently called upon in the course of professional practice, to determine the relative economical value of two forms of furnaces which were in use for burning wet fuel.

The use of fuel, like spent tan or sawdust, actually wet with water or sap, is so unusual, and is so seldom seen by the engineer, that a detailed account of an experimental investigation made upon two distinct varieties of furnace burning spent tan, wet from the leaches, will probably be considered as important and interesting, by the other members of the Society, as it was by the writer.

2. Formerly, it was thought impossible to burn this waste product of tanners, and it was either thrown away, at considerable expense for carting, or was mixed with dry wood or other good fuel at some cost, or it was dried in the open air by the sun, or by artificial heat in kilns. Within a few years, it has been found that with exceptional skill on the part of the fireman or "stoker," it could be burned with some success in furnaces only differing from those of ordinary construction by having a brick arch turned above the grates; in others having "cone" grates with special arrangement and proportions of air space; and with excellent results in a furnace having an overhead brick arch, with a grate so proportioned that a considerable amount of fuel could fall into the ash-pit and burn there, and with the separate "ovens," two or more in number, so arranged that the products of active combustion in one furnace should be mingled *en route* to the boilers, with the products of distillation and with moisture expelled from the fuel, in a similar adjacent "oven," or furnace, which fuel was, at the same time, drying under the heat radiated from the furnace walls and arch, and received from the fire in the ash-pit, thus desiccating preparatory to subsequent combustion. The latter requisite, of preliminary desiccation was secured also by a system of "alternate firing" of the separate feed-holes in the same oven.

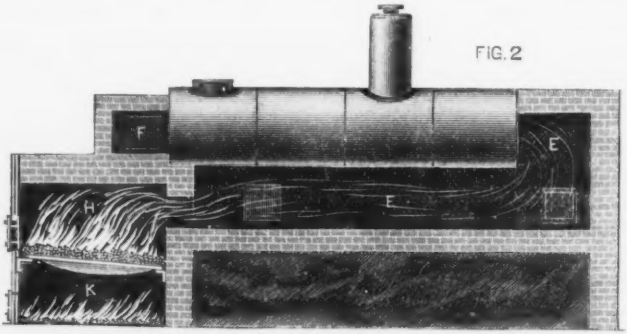
3. The secret of success would seem to be—as indicated by the examination of a large number and of a considerable variety of furnaces burning spent wet tan with more or less success—the surrounding of the mass of wet fuel so completely with heated surfaces and burning fuel

FIG. 1



that it may be rapidly dried, and then so arranging the apparatus that thorough combustion shall be secured, and that the rapidity of combustion be very precisely equal to, and shall never exceed the rapidity of desiccation. Where this rapidity of combustion is exceeded, the dry

FIG. 2

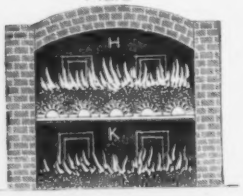


portion is consumed completely, leaving an uncovered mass of wet fuel which refuses to take fire, and then combustion ceased entirely.

In the ordinary steam-boiler furnace, Fig. 1, wet fuel has never, so far as the knowledge of the writer extends, been burned with even ap-

proximate success. Withdrawing the grates from under the boiler, and securing a reservoir and radiator of heat, by throwing over them a brick arch, as in Fig. 2, gives a form of furnace, known either as a Morrison or a Crockett furnace, in which wet fuel has been burned with partial success, and when the grates are so set that some fuel may fall into the ash-pit as it dries, and there burn, the conditions become those of the second case to be described. The use of the form of grate—shown in Fig. 3 (enlarged in Fig. 4)—giving ample space for fuel falling into the ash-pit, ensures still freer combustion. This peculiar form of section also secures freedom from warping when highly heated.

FIG. 3



4. The first example of a furnace burning wet tan—of which the efficiency was determined by the writer, was of the kind known as the "Thompson Furnace," which embodies all of the favorable conditions described in the preceding paragraph. This furnace is shown in section, Fig. 5.

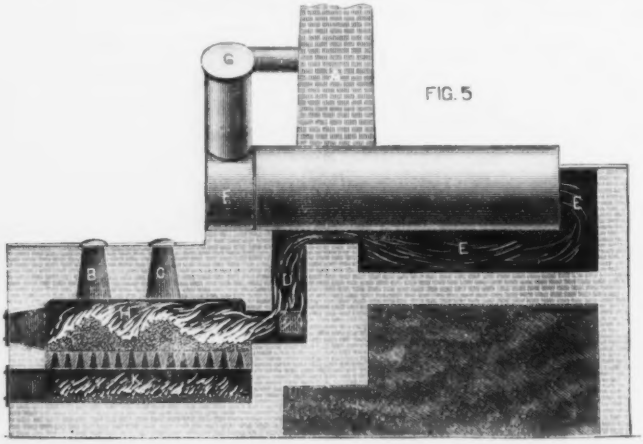


FIG. 5

There were six "ovens" placed side by side, in two sets of three each, the chimney rising between them as shown at *A*. The grate surface of each oven was 9 feet long, and 4 feet 4 inches wide, giving

a total area of 234 square feet. Each furnace was charged through two holes, *B* and *C*, in the top of the furnace arch, the proper method being to fill these holes alternately. The grates were of fine brick, spaced about $\frac{3}{4}$ths inch apart, and supported along the middle line of the ash-pit by a brick wall. The thickness of these grates was nearly 3 inches. They were in four pieces, breadthwise the furnace, a pair on each side the middle wall supported by abutting against each other in a manner somewhat resembling an arch of but two voussoirs. Two doors at the end of each ash-pit permitted cleaning to be readily effected. The gasous products of combustion leaving the furnaces entered a "mixing chamber" *D*, common to each set of furnaces, and thence passed through the flues *E E* to the extreme end of the boilers, returning through the tubes to the smoke-boxes *F*, and through the iron flue *G* to the chimney.

5. The steam-boilers were three in number of the plain multi-tubular variety. Two were 4 feet in diameter, 14 feet in length, and contained 32 four-inch tubes each; the other was 5 feet in diameter, 12 feet long, and contained 78 three-inch tubes. The total heating surface, reckoning all of the tube surface, one-half the surface of the shell and all of exposed surface of the tube plates, was approximately 2 000 square feet. Of this a portion was ineffective, the lower tubes being choked with ashes, and the remainder was partly covered with deposit. The chimney was 90 feet high.

6. The feed-water was heated in a closed heater by the exhaust steam of the engine driving the machinery of the tannery, very nearly, if not quite to the boiling point. The exhaust steam mingled directly with the water. On reaching the tank in which the water was measured the feed water had cooled down to 205° Fahr.; thence it passed through the pump, and a considerable length of pipe across the street separating the engine from the boiler-house, and finally, around to the back end of the boilers, where branch pipes conducted it to each boiler.

The temperature of the feed when entering boilers could not, of course, be determined, but it was probably at least as low as 190° Fahr., which is the temperature assumed in the estimate of efficiency, and it may have been somewhat lower.

7. The measurement of the fuel was made as it lay on the leach, before it was disturbed for the purpose of being removed to the boiler-house, this precaution being taken to avoid error arising from possible changes of volumes due to such disturbance. In one case this transportation to the furnace was performed by a screw, as grain is some-

times moved, and in the other case by cars, capable of carrying nearly a cord, into which the tan was thrown from the leach by hand, and then was dumped at the furnace doors. A careful measurement showed the full leach to contain 8.04 cords of tan. From this 0.34 cord was taken to be weighed after being compressed until it was, as nearly as could be judged, as compact as when in the leach.

8. Commencing at 9 o'clock A. M., the trial continued until the remaining 7.7 cords were burned, closing at 10 o'clock P. M.

9. The feed-water was measured by cutting the feed-pipe between the heater and the feed-pump, and conducting the water from the former into a wooden box 18 inches wide and 4 feet long. The pump drew the water from this box, which ordinarily stood full to the top. Occasionally, the water supply from the heater was shut off, and the time which was occupied by the pump in reducing the level of the water one foot was noted. This was invariably very precisely 4 minutes. The quantity of water pumped into the boiler from the beginning to the end of the trial was 5,625 pounds, or 90 cubic feet per hour.

10. The capacity of the pump was originally barely equal to the requirements of the case, and, at this time, its valves leaked somewhat, making it necessary, not only to run the pump at its full speed throughout the trial, but to keep the fires considerably below their maximum intensity to avoid the necessity of putting on the steam auxiliary to prevent the water getting dangerously low. The maximum evaporation on the trial was thus determined by the capacity of the pump. The flue dampers were kept, as an average, something more than half open. The engineer estimated that he could have burned the fuel with nearly 50 per cent. greater speed, obtaining a proportionally increased evaporation, could the pump have supplied so much water.

11. The spent tan, coming directly from the leach, was so wet as to part with its water when squeezed, wetting the hand. It had been simply drained a few hours, and was as wet as it could be without dripping. The percentage of moisture is given below.

12. As the result of the trial was considered a matter of great importance, both directly and indirectly, it was essential, not only to determine the amount of water entering the boilers, but to determine as accurately as possible the quality of the steam made, thus ascertaining how much water left the boilers as steam, and how much was "primed" over from them unevaporated. As, under the conditions of this trial, each pound of steam obtained from the fuel about eight times as much

heat as was taken up by water "primed" out of the boiler, the serious error which might arise by crediting the fuel with the evaporation of all water entering the boiler, in cases where even a moderate amount of priming occurred, is evident. Instances sometimes occur in which more water passes off unevaporated than is actually turned into steam. At least one instance has occurred in the experience of the writer in which the average amount of water primed exceeded by more than 100 per cent. the weight of steam made, the percentage of priming as the expression is used in this paper, being over 60 per cent. The error arising from the neglect of this circumstance told nearly proportionally in favor of a really very poor boiler. A good boiler ought not to "prime," when working properly, over 10 per cent. when unprovided with superheaters; 5 per cent. priming would represent good work, and as low as 3 per cent. has been attained.

13. The first precise determination of the amount of priming in steam boilers was probably made by the writer in November, 1871, at a competitive trial of 5 steam boilers entered at the Exhibition of the American Institute, the trial being made at the request of a Committee of Judges of which he was then Chairman.*

In that instance, a large surface condenser of about 1 100 square feet area of condensing surface was used, in which all of the steam made by the boiler on the trial was completely condensed. The water fed into the boiler was measured by a meter, and, on issuing from the condenser, was again caught in a tank. The condensing water, which amounted to some 10 tons per hour, was measured by a meter.

The temperature of the injection and discharged water, of the feed, the steam, the water of condensation, and of the escaping products of combustion, were carefully ascertained by suitably arranged thermometers and pyrometers, and were recorded in a log kept by selected students of the Stevens Institute of Technology. The fuel and the ashes were weighed, and all data were obtained and recorded with the greatest possible care.

14. The tabular statement of the results, as given in the report of the Committee, is reproduced as they are not otherwise obtainable by engineers generally, and as they are interesting and valuable in themselves, and particularly, as affording a useful standard with which to compare the results obtained during this trial of tan-burning

* Trans. American Institute 1871-2; Journal Franklin Institute 1872; Van Nostrand's Engineering Magazine, 1871.

15. Results of the Competitive Trial of Steam Boilers at the Fair of the American Institute, November, 1871.

NAME.	Square Feet.		Ratio of heating surface to grate surface.	Total Weights.					Ratio of water primed to water evaporated.	Mean Temperatures.						
	Grate surface.	Heating surface.		Coal.	Combustible.	Feed.	Steam.	Primed water.		Injection.	Feed.	Water of condensation.	Discharge.	Steam.	Superheat.	Flues.
	A.	B.	C.	D.	E.	F.	G.	H.	I.	J.	K.	L.	M.	N.	O.	P.
Root	27	876½	32.5	3 800	3 185.5	27 896	27 896	0.	0.	45°.94	45°.94	58°.31	145°.1	334°.6	16°.08	416°.6
Allen	32½	920	28.5	5 375	4 527	39 670	39 670	0.	0.	45°.5	45°.5	63°.48	154°.76	330°.63	15°.23	345°.87
Phleger	23	600	26.1	2 800	2 274	29 428	19 782.94	645.06	3.26	45°.65	45°.65	54°.38	120°.83	321°.06	0°.	503°.76
Lowe	37½	913	24.2	4 400	3 705	31 000	31 003.35	2 336.65	6.9	45°.0	45°.0	54°.8	131°.5	319°.48	0°.	389°.6
Blanchard	8½	440	51.8	1 232	1 047.5	10 152.5	9 855.6	296.9	3.	44°.4	44°.4	49°.49	106°.14	323°.75	0°.	221°.07

Results, etc.—(Continued.)

NAME.	Total British thermal units.	Total units per pound of combustible.	Apparent Evaporation.				Actual Evaporation.		Equivalent evaporation of water at 212° Fahr. and atmospheric pressure.	Square feet of heating surface required to evaporate one cubic foot of water per hour.	Coal, lbs. per square foot, grate surface per hour.	Efficiency; actual evaporation of fuel divided by theoretical.
			Per pound of coal.	Per pound of combustible.	Per square foot of grate surface per hour.	Per square foot of heating surface per hour.	Per pound of coal.	Per pound of combustible.				
	Q.	R.	S.	T.	U.	V.	W.	X.	Y.	Z.	Z 1.	Z 2.
Root	32 751 834.34	10 281.53	7.34	8.76	86.09	2.65	7.34	8.76	10.64	23.59	11.73	0.709
Allen	46 387 827.1	10 246.92	7.38	8.76	102.51	3.59	7.38	8.76	10.60	17.41	13.88	0.707
Phleger	23 066 685.39	10 143.66	7.26	8.95	73.70	2.83	7.07	8.70	10.49	22.74	10.13	0.699
Lowe	37 228 739.072	10 048.24	7.68	9.12	75.06	3.10	7.20	8.55	10.40	21.63	9.71	0.693
Blanchard	11 485 777.35	10 964.94	8.24	9.69	99.53	1.92	8.00	9.41	11.34	33.48	12.10	0.756

furnaces. The method of determining the percentage of priming will be given below. It should be stated that the Root and the Allen steam boilers were of the "safety" or "sectional" class; the Phleger boiler was also composed of small tubes, but was surmounted by a large drum which was intended to contain some water, thus differing essentially from the two preceding in construction, and in the fact that no portion of the heating surface was above the water-line. The Root and Allen boilers both had a considerable amount of superheating surface. The Lowe and the Blanchard were peculiar forms of multitubular boiler, the former being distinguished by having a peculiarly designed combustion chamber, and the latter by its unusually large proportion of heating surface, as compared with the area of grate, and by its dependence upon a forced draught. All of these boilers gave exceedingly creditable results at this test.

16. A very neat apparatus has been invented by Leicester Allen, of New York, for determining the quality of the steam furnished by a steam-boiler. One of these instruments was made under the direction of the writer, for a committee of the American Institute, and used in 1872, together with the apparatus already described, at the American Institute Exhibition of that year.

17. At the trial about to be described it was impossible to condense all of the steam made, and as no "Allen Calorimeter" was obtainable, it became necessary to improvise apparatus for the occasion. The steam-pipe leading to the engine was tapped by a piece of gas-pipe, on which was fitted a stop-valve. From a short piece of pipe attached to this stop-valve a length of india-rubber hose was led to a convenient point beside the boilers, where a barrel was mounted on an accurate platform scale; 200 pounds of water were carefully weighed into this barrel, and when the scale beam precisely balanced, the weight was set ahead 10 pounds. A very accurate thermometer, which had been provided by the writer, completed this crude yet satisfactory arrangement.

At intervals during the trial the stop-valve was opened, and after allowing steam to blow through the hose freely until all water was expelled, and the hose was so thoroughly heated as to insure that no loss of heat, by the steam flowing through it, should produce condensation and render the results inaccurate, the end of the pipe was plunged into the water contained in the barrel, and the issuing steam allowed to condense until the rise of the scale beam proved 10 pounds of steam to have been added to the weight originally placed in the barrel. The temperature of

the water was carefully observed at the beginning and at the end of the experiment, and the rise of temperature recorded as a basis for the estimates of priming to be given.

18. It was considered advisable to ascertain, if possible, the temperature of the products of combustion escaping to the chimney. No pyrometer was obtainable, and it became necessary to improvise another arrangement for this purpose. A mass of iron, weighing 60 pounds, was found and placed in the flue leading from the boiler, where it, after a time, attained the temperature of the gases flowing past it. A wooden vessel of convenient size and shape was obtained, and 50 pounds of water were carefully weighed into it. At intervals of two or three hours the iron was suddenly removed from the flue and dropped into this water. The initial and final temperatures were noted, and, with the range, recorded for use in calculating the temperature of the waste products of combustion. The pressure of steam was observed hourly.

19. The collated observations gave the following data :

Mean steam pressure during trial			71.4 pounds.
Total amount of spent tan burned			7.7 cords.
" " " water fed to boilers			73 125 pounds.
Temperature of water entering boilers			190° Fahr.
" " in determining priming :			
	Initial.	Final.	Range.
1st observation	60°	110°	50°
2d "	63°	124 (116°?)	53° (?)
3d "	62°	115°	53°
Temperature of water in determining temperature of flues :			
1st observation	65°	119°	54°
2d "	63°	122°	59°
Weight of one cord of wet spent tan, as measured in the leach			5 447.7 pounds.
Length of trial			13 hours.

20. The determination of the total heat derived from the cord of fuel is the first and most important problem. To solve it, it is necessary to know the temperature and weight of feed water, the weight of steam produced and its temperature, the weight of water heated to the temperature of the steam, but not evaporated, and the quantity of fuel consumed. From the data obtained we can readily ascertain the total number of units of heat utilized per cord of wet fuel burned.

21. It is first necessary to calculate what portion of the 73 125 pounds of water passing through the boiler, was actually evaporated. Each pound of steam produced, required for its generation the quantity of heat needed to raise it from the temperature of the feed-water to that due the pressure under which it was formed, and to vaporize it at that temperature. Each pound of water, carried away in suspension by the

steam, only absorbed from the fuel the amount of heat needed to raise its temperature from that of the feed-water to that of the steam.

In heating the water in the calorimeter used in testing its quality, each pound of steam gave up an amount of heat equal to that which would have been required to raise its temperature from that of the mass in the calorimeter at the end of the experiment, to that of the steam in the boiler, and to evaporate it at the latter temperature and pressure.

Each pound of water entering the calorimeter, surrendered a quantity of heat equal to that needed to raise its temperature from the final temperature of the calorimeter to that of the steam under boiler pressure.

22. The total amount of heat being the sum of these two quantities, we may construct an algebraic equation which shall embody all the conditions of our problem.*

Let H = the number of heat units per pound of steam, h = the number of heat units per pound of water, U = total heat transferred to calorimeter, W = total weight of steam and water, x = total weight of steam alone, $W - x$ = weight of water alone.

$$\text{Then } Hx + h(W - x) = U; \text{ or, } x = \frac{\frac{U}{h} - W}{\frac{H}{h} - 1}.$$

23. At the first experiment, the steam pressure, per gauge, was 75 pounds. The temperature of steam at this pressure is 320° Fahr. The "total heat" of steam at 320°, from 0°, and at 75 pounds pressure, is $(320—212)\ 0.305 + 212 + 66.6 = 1\,211.5$.

The heat transferred to the calorimeter, per pounds of steam, was therefore $1\,211.5 - 110 = 1\,101.5$ thermal units in this experiment.

The heat transferred, per pound of water, was $320—110 = 210$ thermal units.

The total quantity of heat transferred to the 200 pounds of water, by 10 pounds of mingled steam and water, was $200\ (110—60^{\circ}) = 10\,000$ thermal units.

$$\text{Finally, } x = \frac{\frac{10\,000}{210} - 10}{\frac{1\,101.5}{210} - 1} = 8.87 \text{ pounds steam.}$$

$$W - x = 10 - 8.87 = 1.13 \text{ pounds of water.}$$

The percentage of priming was, therefore, 11.3. The ratio of weight of

* See Report of Committee on Test of Steam Boilers: Trans. Am. Inst., 1871-2.

steam and water was $\frac{8.87}{1.13} = 7.85$, the water being $\frac{1.13}{8.87} \times 100 = 12.74$ per cent. of the steam.

24. The other experiments were made with the steam pressure as before, and, in the second, the valve of $W - X$ comes out negative, indicating superheating. This may, possibly, have actually occurred as a consequence of the water having fallen slightly below the upper row of tubes in one boiler, but it is more probable that the reading, 124°, does not represent the mean temperature of the mass of water in the calorimeter. In this experiment, the water was not as carefully stirred with the thermometer as in the other experiments, and the temperature was taken at the surface of the water, after a first and otherwise satisfactory reading of 116° had been obtained, but a second application of the steam jet had been necessary, to accurately balance the scale, which heated the surface above the average temperature of the mass previously heated. The true reading can probably have been no higher than 116° or 117°, and it is taken for purposes of calculation at the former figure, although the lowest unrecorded reading, finally, actually obtained at the middle of the well-stirred mass was 116°.

$$\text{Then } x = \frac{\frac{10\,600}{204} - 10}{\frac{1\,095.5}{204} - 1} = 9.6 \text{ pounds steam,}$$

and the weight of water being $10 - 9.6 = 0.4$, the percentage of priming was 4, and the water carried over weighed $\frac{0.4}{9.6} \times 100 = 4.3$ per cent. as much as the steam with which it was mingled.

In the third experiment

$$x = \frac{\frac{10\,600}{205} - 10}{\frac{1\,096.5}{205} - 1} = 9.59\,; \quad W - x = 0.41.$$

The percentage of priming was 4.1, and of water to steam 4.2 per cent.

The mean percentage of priming was 6.47. The mean percentage of steam alone was 93.53.

25. The total quantity of heat derived from the fuel and taken up by the boilers can now be divided into two portions and each calculated.

The total weight of steam produced was $73\,125 \times .9353 = 68\,393.8$.

" water primed " $73\,125 \times .0647 = 4\,731.2$.

The mean pressure at which this steam was formed being 71.4 pounds, we find its "total heat" per pound to be 1 210.6 thermal units, and the heat communicated to each pound of feed entering at 190°, and evaporated at this pressure, 1 020.6 units. The average heat received from the fuel by each pound of water not evaporated was 127 thermal units.

Then 68 393.8 × 1 020.6 = 69 802 712.3 units,
and 4 731.2 × 127.0 = 600 862.4 "

Total heat from the fuel = 70 403 574.7 thermal units.

$$\text{Total heat per cord tan } \frac{70\,403\,574.7}{7.7} = 9\,143\,321.4 \text{ units.}$$

26. The usual standard, as generally accepted by engineers in examples of this kind, is the evaporation of one pound of water, at the boiling point, and under atmospheric pressure.

The heat required is the latent heat at 212°, or 966.6 thermal units per pound. We have, therefore—

Equivalent evaporation, by one cord of wet spent tan, from 212°, under atmospheric pressure :

$$\frac{9\,143\,321.4}{966.6} = 9\,459.2 \text{ pounds of water.}$$

27. Under these conditions, 10 pounds of water would be considered a fair evaporation per pound of good coal, and in this example therefore, the furnace utilized from each cord of tan the equivalent of 946 pounds of coal.

28. A quantity of the tan was placed in a "fruit jar," and hermetically sealed. This tan was carefully weighed by Prof. Geyer, at the Stevens Institute of Technology, dried by exposure to the air in the study of the writer for one week, and then again weighed by Prof. Geyer, in the presence of the writer. The weights, before and after drying, were respectively 656.8 grammes, and 268.8 grammes. This fuel contained, therefore, 59 per cent. water, and but 41 per cent. woody fibre.

The weight of a cord of this tan, measured in the leach, and then well dried in the open air, would be 2 233.56 pounds, and the equivalent evaporation per pound becomes 4.24 plus that of the water contained in the fuel, say 1.44, or 5.68 pounds water per pound of combustible.

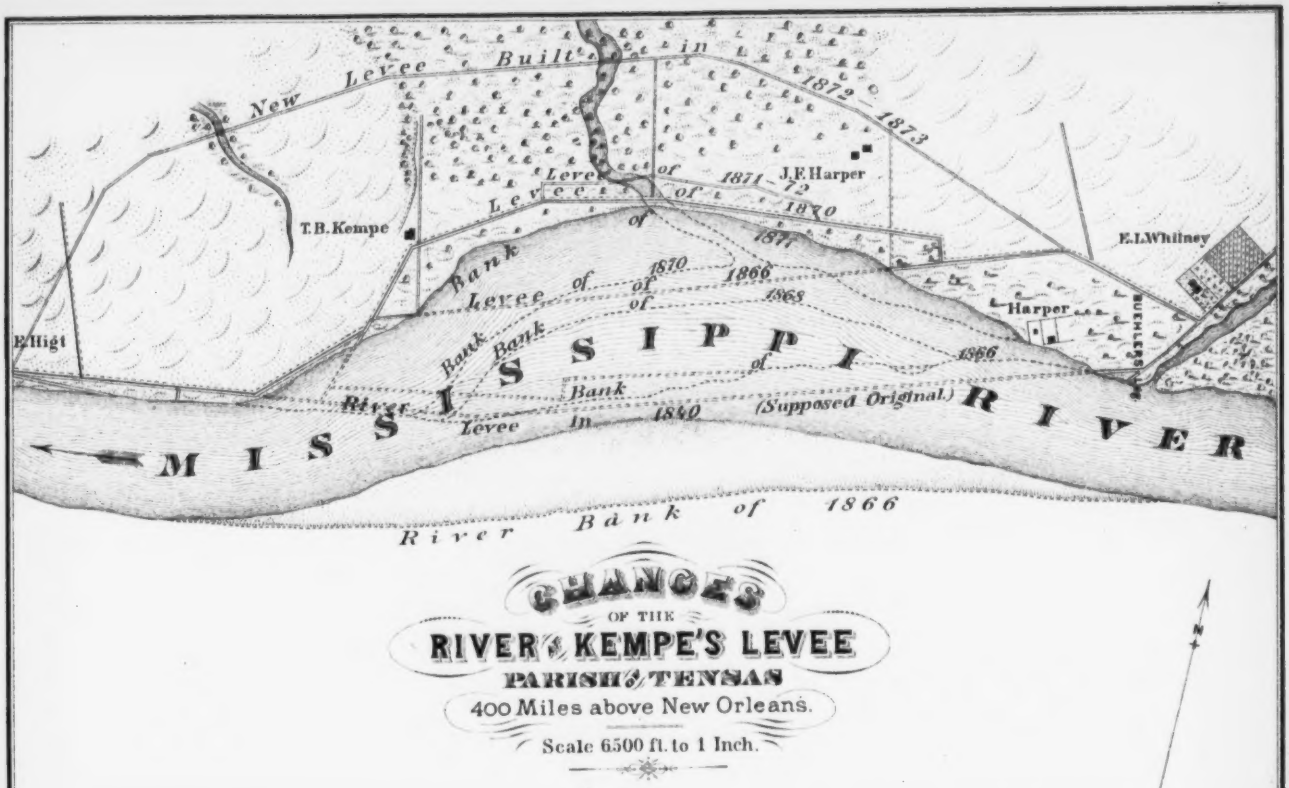
29. The determination of the temperature of chimney flue, or of the escaping gaseous products of combustion, is thus made. At the first observation, 50 pounds of water were heated from 65° to 119° Fahr., a range of 54°, by the cooling of a mass of iron weighing 60 pounds, from

an unknown temperature to 119° Fahr. The amount of heat communicated to the water was $50 \times 54 = 2\,700$ thermal units. Each pound of iron, therefore, parted with $\frac{2\,700}{60} = 45$ units of heat.

The specific heat of iron is given by Watts as 0.112. It requires, therefore, the cooling of one pound of iron through 9° of temperature to heat a pound of water one degree. The iron, in the case considered, must therefore have lost $45 \times 9 = 405$°, when cooled to 119°, and its original temperature, and that of the escaping gases in the flue, must have been $405° + 119° = 524°$. The second observation, in a similar manner, gives the temperature of the chimney flue at 564.5°.

Watts gives 315° Centigrade, 599° Fahr., as a proper temperature with natural draft. Rankine gives absolute temperature of external air multiplied by $\frac{25}{12}$ as the temperature giving most effective draught. In this case, therefore, in which the average temperature of the air was 74°, the best temperature of chimney would have been $\left(25 \frac{74 + 461}{12}\right) - 461 = 645°$.

30. The minute inaccuracy of the results thus obtained, which is due to changes of the specific heat of water, and of metal, under varying temperatures, is of no practical importance. As the vessel containing the water heated was of wood, in each case, the usual correction for heating the vessel when metallic becomes of no importance also, and the weight of the thermometer being insignificant in comparison with that of the water, that correction is unnecessary. This method is of great value as a last resort, in absence of other good heat measuring appliances.


New Levee Built in 1872 - 1873
Levee of 1871 - 72
Levee of 1870
Bank of 1871
Levee of 1870
Levee of 1866
Levee of 1865
Bank of 1866
Bank
Bank
River Bank of 1866
Levee in 1840 (Supposed Original.)
MISSISSIPPI RIVER
River Bank of 1866
T.B. Kempe
J.F. Harper
E.L. Whitney
Harper
E. Higt
CHANGES
OF THE
RIVER & KEMPE'S LEVEE
PARISH & TENSAS
400 Miles above New Orleans.
Scale 6500 ft. to 1 Inch.
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PART II.*

31. The second furnace which was experimented upon by the writer, was of the form known as the "Crockett." This form of furnace is shown in Fig, 2, and that here described was of the same general form as that illustrated, differing principally in its arrangement of bridge walls.

In this example two furnaces were constructed side by side, each having a grate surface of 4 x 6 feet, the total grate area of both being 48 square feet.

The grates were of cast-iron, of ordinary form, set so closely that none of the wet fuel could fall through into the ash-pit. It was stated that it was not intended that the charred fuel should fall through and burn in the ash-pit.

During this trial, however, more or less burning tan was continually falling into the ash-pit and burning there. This, undoubtedly, assisted, in some degree, in the desiccation of the wet fuel, by direct radiation of heat, and by heating the entering air as it passed over this bed of hot coals. To this extent the furnace resembled, in its action, the Thompson furnace, already described.

Above the grates a brick arch was turned, as shown, against which the products of combustion impinged and the heat radiated from the burning fuel on the grate, keeping this arch at a high temperature, it assisted the process of desiccation of wet fuel, when first thrown in, by strongly radiating upon it the heat thus stored up while the fires were most intense. From the furnace the gases passed directly into the flues beneath the boiler.

32. The tan, wet from the leach, was charged into the furnaces through the doors in the front, as in all usual forms of furnace, and the process of "firing" differed but little from that usual with thin fires, where coal is used. The fuel was thrown in at intervals of between 5 and 8 minutes, the furnace-man taking care, first, to fill all holes in the burning mass, and, next, covering the whole with a very evenly distributed and very thin layer of fresh fuel. This fresh charge was quickly dried by the heat of the burning fuel, over which it was spread, and by the heat radiated from the hot furnace arch above it, and, taking fire, burned freely. No special effort seemed to be made to obtain "alternation" in working the furnace, but the irregularity with which

* Transactions CII. "Efficiency of Furnaces burning Wet Fuel," by Prof. Robert H. Thurston, continued.

the fuel burned at different parts of the grate did, perhaps, secure something of this effect.

33. The fuel was measured in the leach, as in the previous case, and about a half leach, measuring 4½ cords, was burned during the trial, between 8 o'clock A. M. and 6 o'clock P. M. The actual working time was 9 hours, the work being stopped from 12 M. to 1 P. M.

34. The tan bark burned was hemlock, mixed with some oak. It looked like a better material than that used in the other trial. It was more cleanly ground, seemed less "soggy," and had a much better color.

35. The boilers used here were two in number. One was an old-fashioned "Cornish" boiler, 4 feet in diameter and 18 feet long, with one 24-inch flue. The second was 6 feet in diameter, with four 18-inch flues; the total length was 15 feet. The gases from the furnace were led under the boilers, and then, with a double return through the boiler-flues to the chimney. The total heating surface, reckoned as before, was very closely 700 square feet.

36. The flues were stated to have been so long in use without cleaning, that the draught was somewhat impeded by the accumulation of ashes beneath the boilers, and that the rapidity of combustion was somewhat lessened. The two trials are, therefore, both to be taken as representing less than the maximum capacity of the furnaces.

37. The trial was made by a somewhat similar method to that adopted in the one already described. The quality of steam was determined similarly. The water was measured differently. In this case the capacity of the feed-pump exceeded several times the requirements of the boiler, and the only absolutely reliable means of determining the quantity of fuel seemed to be to measure every pound going to the pump, thus evading the uncertainty attending any attempt to secure regularity in the action of the latter.

38. A barrel was fitted to the suction-pipe of the pump, and an employé of the Mechanical Laboratory of the Stevens Institute of Technology was stationed with a hose to fill it with water, when it became empty, and to keep an account of the number of barrels used, while an employé of the proprietors of the tannery, stationed at the pump, checked the account. All of the water fed into the boilers during the trial was thus measured, and at the close of the trial the barrel was taken out, filled on the scales, and the weight of its contents determined.

39. There was no opening into the chimney flue through which the escaping gases could be reached, and their temperature was not determined.

40. The amount of smoke issuing from the chimney of this furnace was considerably greater than was observed at the preceding trial, indicating that the Thompson Furnace secured a somewhat more perfect combustion than the Crockett.

41. The results of this trial gave the following data :

Total number of cords of tan burned..... 4.5
" weight of water fed to boilers..... 28 509 pounds
Temperature of feed-water entering boilers (estimated)..... 160° Fahr.
" water observed in determining "priming" :

	Initial.	Final.	Range.		Initial.	Final.	Range.
1st. observation.....	68°	118°	50°	3d observation	76°	126°	50°
2d "	70°	118°	48°	4th "	96°	124°	48°

Length of trial..... 9 hours.

42. Estimating the priming as before, we obtain from the several observations, which were made with the steam pressure, per gauge, 55, 50, 45 and 60 pounds, respectively, $x = 9.02$, $x = 8.07$, $x = 9.12$, and $x = 8.55$, and the per centage of priming, 9.8, 19.3, 8.8, 14.5 per cent. The mean of all observations gives the average percentage of priming at 13.1 per cent., and indicates that the steam issuing from the boiler carried in suspension 15 per cent. of its own weight of unevaporated water.

43. We determine the total heat from the fuel thus :

Steam produced..... 28 509 × 0.869 = 24 774.32 pounds
Water primed..... 28 509 × 0.131 = 3 734.68 "

The mean pressure of steam during the trial was 50.44 pounds per square inch, and its temperature 298° Fahr. Its total heat at 298° from 0° was 1 204.8 units per pound. Then we have

Total heat, per pound of steam..... 1 204.8 − 160 = 1 044.8
" " water..... 298 − 160 = 138 , and
Total heat transferred from fuel to steam..... 25 884 269.54 units.
" " " water..... 515 385.84 " and hence
Total heat transferred from fuel to feed..... 26 399 595.38 units.
" per cord of wet tan..... 6 866 576.75 "
Equivalent evaporation per cord, water at 212°..... 7 103.84 pounds.
" weight of coal per cord of tan..... 710.38 "

44. A sample of the fuel used in this trial was sealed up, as before, and was similarly weighed at the Stevens Institute of Technology, dried in the air one week, and its loss of moisture determined.

This sample contained 55 per cent. of water, and 45 per cent. ligneous material. The weight of the tan in the leach was judged to be practically the same as in the preceding trial, and the equivalent evaporation per pound is $\frac{7\,103.84}{2\,233.56} = 3.19$ plus the water held by the fuel, say 1.22, or 4.41 pounds.

45. Comparing the quantities of heat actually utilized by transfer from the fuel to the boiler, we obtain as the measure of the actual comparative efficiencies of the two furnaces $4.24 \div 3.19 = 1.33$, the Thompson excelling the Crockett 33 per cent. A number of circumstances combine to make the actual difference somewhat greater than the record here indicates, but this result may probably be taken to represent practically the relative economical standing of the two furnaces.

46. Experiments on dry pine wood, made by Prof. Johnson during his extended and invaluable examination of American coals,* for the U. S. Navy, furnish a standard of comparison which will be useful here. One cord of well-seasoned yellow pine wood weighed 2689.2 pounds—10 per cent. more than a cord of thoroughly air-dried spent tan bark—and one cubic foot weighed 21 pounds. Experiments on evaporative power showed, as a mean result, an effect equivalent to the evaporation of 4.69 pounds of water from 212°, under atmospheric pressure, per pound of wood consumed, the temperature of chimney flue being 315.2°, and the wood burning at the rate of 15.87 pounds per square foot of grate per hour, under a boiler having a ratio of heating to grate surface of 26.83 to 1.

47. Comparing this result, as a standard, with the evaporation obtained in the two wet fuel furnaces, per pound of fuel, exclusive of water, we get for the Thompson furnace $\frac{5.68}{4.69} = 1.21$, and for the Crockett, $\frac{4.41}{4.69} = 0.94$. The Thompson furnace is thus seen to have given a better result per pound of ligneous combustible when burning wet tan than was obtained in the ordinary steam-boiler furnace, burning seasoned yellow pine, this superiority reaching 21 per cent. The Crockett furnace had 94 per cent. of the efficiency of the common wood-burning steam-boiler furnace.

48. The relative efficiency of fuel, comparing wet tan with dry wood, by weight, the former containing between 55 and 60 per cent. of

* Report to the Navy Department on American Coals. By Walter R. Johnson. Washington, 1844, pp. 546–550.

water, becomes $\frac{4.24}{4.69} \times 100 = 90.4$ per cent., each fuel being consumed under the most favorable conditions shown above, and equal weight of ligneous fuel being taken for comparison.

A cord of dry yellow pine, as per experiments of Prof. Johnson, evaporated 12 618.3 pounds of water. A cord of wet spent tan, burned in the Thompson furnace was equivalent to $\frac{9\,459.2}{12\,618.3} = 0.75$ cord of dry wood. One cord of wet spent tan burned in the Crockett furnace, was equivalent to $\frac{7\,103.84}{12\,618.3} = 0.56$ cord of dry yellow pine.

49. A cord of dry yellow pine is approximately equal in heating power to 0.6 of a ton of coal, and, conversely, the ton of good coal is equal in calorific power to 1.66 cords of soft wood. An average pound of dry wood is theoretically capable of evaporating 6.66 pounds of water from and at 212°. A pound of good anthracite, similarly, should evaporate 13.5 pounds of water.

The "absolute efficiencies" of coal and wood, under the conditions already described in the several cases mentioned, are as follows: coal 70 per cent.; wood—in the Crockett furnace, 66 per cent., in the ordinary steam-boiler furnace, 70 per cent, and in the Thompson furnace, 85 per cent.—reckoning the evaporation of the moisture in the fuel. Excluding this moisture, the percentages become respectively 70, 48, 70 and 64.

50. The data obtained at these trials are sufficiently complete to furnish a basis upon which to construct the theory of action of each furnace, and to give approximate determinations of quantities which are of importance in that connection, and of interest in their bearing upon practical deductions. The most important points are the temperatures of furnace and of chimney flue, the quantity of air supplied, and the effect of variations of area of heating surface of boilers.

51. An approximate determination of the temperature of furnace can be made, in this case, in the following manner:

The fuel used at this furnace, as taken from the leach, contained more than one-half its weight of water. In handling, it lost some of this moisture. When thrown upon the top of the furnace, and before it was thrown upon the fire, it, by its non-conducting property, prevented to some extent loss of heat, and such heat as was absorbed by it was usefully employed in evaporating its moisture. The quantity of water thus lost before entering the furnace may be estimated approximately at about

1 per cent. in handling, and 3 per cent. at the furnace. Box, in his "Treatise on Heat," gives the total loss of temperature by conduction and radiation, in a fire-brick furnace, as 10 per cent. Here, several long ovens were placed side by side and the principal loss was that from the top. Very little could take place laterally, and no heat could pass downward.

The total loss here may be taken as $2\frac{1}{2}$ per cent., which would so change the composition of the wet fuel as to leave it with 3 per cent. less water, making it about 45 per cent. combustible and 55 per cent water.

Taking the available heat per pound of the dry portion, at 6 480 thermal units, each pound of wet fuel yields 2 916 units of heat. Of this, 531.6 are absorbed in the evaporation of the 55 per cent. of water, leaving 2 384.4 units to raise the temperature of the products of combustion. Of these there are, as a minimum, 3.7 pounds having a mean specific heat of about 0.287.

The elevation of temperature is therefore 2 245.3°, and adding the mean temperature of the atmosphere, 74°, the mean temperature of furnace, assuming no dilution with unused air and no losses, would have been about 2 320°. Losing about $2\frac{1}{2}$ per cent., the temperature becomes 2 260°.

The temperature of chimney flue was found by experiment to have been 544°. The furnace gases were therefore cooled 2 260°—544° = 1 716° by the loss of the heat given up to the boiler. This is equivalent to $1\,716 \times 0.287 = 492.5$ heat units, per pound of gas, and to 4 049.4 units per pound of ligneous material in the fuel.

The "equivalent evaporation," from and at 212°, is $4\,049.4 \div 966.6 = 4.18$ pounds of water. The actual evaporation was equivalent to 4.24 pounds, and the difference—less than one per cent.—represents losses and errors of approximation.

52. The actual existing temperature of furnace can be thus estimated: the available heat per pound of fuel, excluding water, has been given at 2 916 thermal units. Of this $\frac{531.6}{2\,916} = 0.182$ was not useful in raising the temperature of either the furnace or the chimney. Hence, of all heat liberated, $1.00 - 0.182 = 0.818$* was efficient in elevating the temperature of furnace, and $0.37 - 0.182 = 0.188$ was effective in producing the observed temperature, 544° of chimney. Then, since the same quantity of gas passes at both places, the temperature of furnace was

* Assuming 6 480 units as the heating power of the tan when dried, the efficiency of this furnace becomes 0.63, and 0.37 of the total heat of the fuel passes off by the chimney.

$\left(\frac{0.818}{0.188} \times 470\right) + 74^\circ = 2\,118.5^\circ$. To this is to be added the slight loss of temperature, *en route* between furnace and chimney, by conduction and radiation.

53. The air supply of the Thompson furnace is unusually restricted, as has already been noted.

Instead of an ash-pit with a front entirely open, we find here closed ash-pit doors, and no other opening for the passage of air than the comparatively small orifices in the registers with which the doors are fitted. The usual amount of air supplied, in furnaces burning coal, is generally given as about twice the theoretically required quantity, giving a temperature of about 2 400°. In exceptional cases, the air supply falls as low as one and a half times the theoretically required amount, the temperature reaching about 3 000°. In such cases, it sometimes happens that the grates are melted down, cast-iron melting at between 2 700° and 2 800°.

In the Thompson furnace, with an ash-pit fire, this is very likely to take place with iron grates, and the inventor was therefore driven to the use of fire-brick grates. The mean temperature of the products of combustion is, however, lower, notwithstanding the restricted air supply, in consequence of the presence of moisture.

The quantity of air supplied is calculated as follows: the difference between the theoretical and the actual temperature of furnace, as above estimated, is $2\,260^\circ - 2\,118^\circ = 142^\circ$, corresponding to $142 \times 0.287 = 40.75$ thermal units per pound of gas, or $40.75 \times 3.7 = 150.8$ units per pound wet fuel, and $150.8 \div 0.45 = 335.1$ units per pound of wood, which heat was distributed through the air diluting the products of combustion.

This latter amount is sufficient to heat $(335.1 \div 0.238) \div 2\,044 = 0.69$ pounds of air from the temperature of the external air 74° to 2 118°. The theoretically required quantity of air, per pound of wood, is 6 pounds; hence, the products of combustion, at the Thompson furnace were diluted with 12 per cent. of air; this is one-fourth that of ordinary practice with coal fires.

54. The temperature of chimney flue not being known, the estimates of temperature of furnace and of air supply cannot be so satisfactorily determined for the Crockett furnace. The following may, in the opinion of the writer, be taken as a fair approximation.

The composition of the fuel, in this case, was slightly changed in handling, between the leach and the furnace, as before. Lying in front

of the furnace also, a small amount of drying must have occurred by radiated heat. Taking the total amount of both influences as producing an alteration of 2 per cent. in the composition of the fuel, it becomes 52 per cent. water and 48 per cent. dry wood.

55. The theoretical temperature of furnace gases, estimated as before, is reduced largely in this case, by the air of dilution. The fuel was burned at the rate of about 20 pounds per square foot of grate per hour, and the weight of carbon, the only valuable heat-producing element, was 5 pounds per square foot of grate per hour. The bulky character of the fuel compelled the opening of the doors, in charging the furnace, about once in seven or eight minutes. The fire burned somewhat irregularly into holes, allowing an unusual amount of air to pass up unutilized.

In ordinary practice, with anthracite coal fires, the fuel burns, with such draught as was found at the Crockett furnace, at the rate of about 10 pounds carbon per square foot of grate per hour. The doors are opened about once in fifteen or twenty minutes, and the air supply is about 240 pounds per square foot of grate surface per hour.

56. The air supply, at the Crockett furnace, was apparently in excess of that just given, by a large amount. Neglecting this excess, and calculating the temperature of the furnace as if the air supply were the same as with coal, the following results are obtained :—

Available heat per pound wet tan	3 110.4 units.
Rendered latent by evaporating 52 per cent. water	503.6 "
Effective in elevating temperature	2 606.8 "

This was distributed through 12.5 pounds of gaseous products of combustion, of a mean specific heat, 0.25. The elevation of temperature was therefore 832.8°, and, adding the mean temperature of external air, we obtain, for the average temperature of gases escaping from the furnace, including cold air streaming through the furnace doors and the holes in the fire, 919.3°, or about 100° above the temperature at which combustible gases can take fire.

Taking the air supply as possibly, at times, as low as that considered the minimum with ordinary coal fires, 180 pounds per square foot of grate per hour, where burning fuel with sluggish draught, the temperature of furnace at such times becomes 1 150.3°.

57. The temperature of chimney flue may be readily estimated for these conditions. Referring the performance of this furnace, like the preceding, to "dry wood" as a standard, its efficiency is 47 per cent. Hence 53 per cent. of the heat obtainable from the fuel passes off by the chimney. The loss of heat by the escape of unburned carbon, in the

form of smoke, although more than at the other furnace, was not probably sufficiently to be here taken into account. The following are the estimates :—

Available heat per pound of wet tan	3 110.4 units.
Passing up chimney, 53 per cent.	1 648.5 "
" " latent in vapor	503.6 "
" " elevating temperature	1 144.9 "

This was distributed throughout 12.52 pounds of gas, where the usual supply of air was maintained, and through 9.64 pounds in the case of less free supply, supposed possible at times.

The heat per pound of gas amounts to 91.4 units for the first case, elevating the temperature $91.4 \div 0.25 = 365.6°$ above that of the atmosphere, or to 452°.

Under the other supposed conditions, the elevation of temperature of chimney becomes 467.7°.

To secure an economy, at this furnace, equal to that obtained at the Thompson furnace, it would be necessary to reduce the quantity of heat carried off by the chimney gases, in the proportion of 53 to 37.

One-third of this heat is latent in the vapor of water, and no part of that can be secured. The requisite reduction of temperature of gases to effect this economy is thus exaggerated ; and it would be necessary, were such a thing possible, to bring down the temperature of chimney to between 140° and 160°, to a temperature 160° or 140° below that of the boilers itself, or to less than one-fourth that required for most efficient draught.

58. In determining the precise value of two competing sets of apparatus, as generators of heat, it is necessary first to obtain from each its best performance in producing heat, and then to provide means of absorbing and utilizing that heat with equal thoroughness in both cases.

The proper area of heating surface of boilers will therefore vary with each case. The same area, where the furnaces themselves differ considerably, may, in the one case, allow a waste of heat, while in the other it may reduce the temperature of gases so far as to compel the adoption of a "mechanical draught." In the cases here considered, the area of heating surface was, fortunately, very nicely adapted, in each instance, to the requirements of the case. In both examples, the temperature of chimney flue is found to be somewhat below that required for most efficient draught ; but was not far different in the two cases, the less economical furnace having the economical advantage of such difference as did exist.

The trial, therefore, exhibits very fairly the intrinsic values of these

two furnaces, as heat-generating apparatus, and of these two radically different methods of working them, as taken apart from the efficiency of heat-absorbing or heat utilizing contrivances.

59. The importance of high temperature of furnace is strikingly and beautifully illustrated by these results. The two furnaces develop practically the same amount of heat from the fuel, but the one distributes it through a large volume of gases at low temperature, sending a considerable proportion of it up the chimney, while the other raises a small volume of gas to a much higher temperature, making it more available to the extent of 33 per cent., and finally, even then, sending up the chimney gases of higher temperature than the first.

The abstract *efficiency of the furnace*, in any ordinary case, is represented by the formulæ—

$$e^1 = \frac{t_1 - t_2}{t_1 - t_3} = \frac{T_1 - T_2}{T_1 - T_3}.$$

Where e^1 represents the efficiency and t_1 and t_2 are the absolute temperatures at which the heat is generated, and at which wasted heat is discharged, and t_3 that of the external air, T_1, T_2, T_3, are temperatures on the Fahrenheit scale.

In these cases $e^1 = \frac{2\,118^\circ - 544^\circ \text{ F.}}{2\,118^\circ - 74} = 0.77.$

$$e^{1\,1} = \frac{919^\circ - 452^\circ}{919^\circ - 86.5} = 0.56.$$

60. These values do not represent the *efficiency of the fuel*, including the vaporization of water contained within itself. In these cases the heat lost, as latent in vaporization, before the generation of these temperatures, is to be deducted to give the total efficiencies of the fuel, which thus are found to have the values 0.77 (1—0.182) = 0.63 and 0.56 (1—0.175) = 0.46.

The experimental determinations were 0.64 and 0.48, if referred to seasoned wood, and 0.63 and 0.47 when referred, as here, to dry wood of a calorific value of 6 480 heat units.

To make the values of e comparable with the standard already assumed for *final absolute efficiency*, per experiment, it is necessary to add 9 per cent. to the first values of e, in each case, in order to credit the fuel with the heat used in the vaporization of its water, and with the heat carried by the vapor up the chimney.

Thus the values $0.77 + 0.09 = 0.86$, and $0.56 + 0.09 = 0.65$ are deduced. These ratios, by experimental determination, were 0.86 and 0.67. The correspondence of these figures with those just deduced theoretically, is a

remarkably conclusive evidence of the accuracy of the estimated temperature and of the fact that the difference of efficiency found by trial is due to such difference of temperature. The accordance is unusually precise.

61. Rankine has given a formula* for determining the efficiency of fuel in ordinary steam boiler practice, where the ratio of the area of heating surface, and of fuel burned per hour, to the square foot of grate surface, is known :—

$$\frac{e^1}{e} = \frac{B\,S}{S + A\,F},$$

in which $\frac{e^1}{e}$ is the quantity called above e^1, A and B are constants, and F and S are the ratio of fuel burned per hour to the square foot of grate, and the ratio of area of heating surface to grate area.

For the cases here considered, $A = 0.5$, and $B = 0.92$, $S = 8.5$ for the Thompson, and 14.5 for the Crockett furnace, $F = 1.38$, and 5.3 ; $\frac{F}{S}$ becomes 0.13 and 0.36, and the value of $\frac{e^1}{e}$ is, for the Thompson 0.859, and for the Crockett furnace 0.776.

Were this formula applicable to these cases, the experimental determinations of efficiency should coincide with these, but they are 0.64 and 0.48. This difference is a consequence of the facts that the fuel used in these furnaces was wet, and that the large proportion of heat absorbed by the water was so much abstracted from the efficiency of the fuel. Thus the absolute values are reduced.

They differ, also, in consequence of the important fact exhibited in the preceding paragraphs, that the temperature of furnace differs in each case (and in the case of the Crockett furnace immensely), from the temperature, 2 400° given by Rankine as the mean temperature of furnaces to which his formula is adapted. This changes relative values.

The value 0.859, for the Thompson furnace, is almost precisely that obtained by experiment—"including water in fuel," 0.86—as it should be, since the temperature of that furnace, 2 118°, is not far different from that of the ordinary coal fire. The value, 0.776, for the Crockett furnace differs greatly from that formed by experiment—"including water in fuel," 0.67—as would be expected in consequence of the exceptionally low temperature of that furnace.

The difference between these two theoretical values, $0.859 - 0.776 = 0.083$, would represent approximately the loss of total absolute efficiency that might be expected, were the case one of ordinary practice, and were

* Steam Engines and Prime Movers, p. 292, § IV.

the Thompson furnace supplied with boilers of as small a ratio of heating surface to fuel consumed, $\frac{F}{S}$ as the Crockett furnace actually had.

The difference which would really be produced would be less in consequence of a circumstance, peculiar to that example, of which the influence has not been noticed by writers on this branch of the theory of engineering.

62. The rate of conduction of heat from the furnace gases to the heating surfaces with which they are in contact varies, in some not well determined ratio, with the difference of temperature. It may be represented approximately, according to experiments of Charles Wye Williams* and, judging from the analysis of M. Paul Havrez,† by a hyperbolic curve, of which the equation is $x\ y = A$; y representing the evaporation for a unit of area of a tube at a distance x from the furnace.

$U = B \log x$ is the equation of total evaporation.

When the volume of gas is the same, as in cases to which the formula of Rankine applies, the constants in these equations are the same, and his formula gives a remarkable satisfactory approximation. Where, as in the Thompson furnace, the restriction of the air supply causes a comparatively slow movement of gases along the heating surface, the value of that portion nearest the fire becomes enhanced, leaving the furthest portions of less efficiency.

The effect of reducing the air supply nearly one-half would, therefore, be to actually reduce greatly the amount of the theoretical loss, 0.08, just given. The real loss would be somewhere between this 8 per cent. and the smaller differences noticed between the theoretical estimates of efficiency of fuel and the actual differences shown by experiment. This latter consideration may, perhaps, be taken as a proof that this difference of efficiency due to such a change of heating surface, would amount to approximately 2 per cent. Were more steam wanted, this would be at once sacrificed at the Thompson furnace, to bring the temperature of chimney up to that, 645° which would give most efficient draught.

63. At the Crockett furnace, the effect of the exceptionally low temperature of furnace is to equalize the value of heating surface; and the considerable velocity of the gaseous current, which is a consequence of the unusually great volume of air passing through the furnace increases

* "On the Steam Generating Power of Marine and Locomotive Boilers."—London, 1864.

† "Evaporisation decroissante en Progression Geometrique dans les Chaudieres."—Revue Industrielle, 1874.

Results of Trials of Furnaces burning Wet Spent Tan Bark, August, 1874.

Kind of Furnace.	Fuel.	Percentage		Total weights pounds.			Steam.	Priming	Percentages.		British thermal units per cord of tan.	Apparent evaporation.				Actual evaporation.		Ratio of Air supply to theoretical.
		Water.	Combustible.	Fuel.	Combustible.	Feed.			Priming to feed.	Priming to steam.		Per cord fuel.	Per pound combustible.	Per sq. ft. grate per hour.	Per sq. ft. heat surf. per hour.	Per lb. of fuel.	Per lb. combustible.	
Thompson....	Tan.....	59	41	44945	17198.41	73125	68393.8	4731.2	6.47	6.92	9143321.4	9596.75	4.25	23.7	2.81	1.3	3.98	1.12
Crockett.....	Tan.....	55	45	22336	10051.20	28509	24774.32	3734.68	13.1	15.	6866576.75	6335.33	2.83	65.9	4.52	.....	2.46	8.
Ordinary steam boiler	Pine wood.	....	80	.....	2360.50	.....	.....	.....	.....	.....	.....	10891.26	4.05	61.5	2.29	4.05	4.03	.....
Ordinary steam boiler	Anthracite Coal.....	....	91	.....	.....	.....	.....	.....	.....	6	.....	.....	8.75	95.	3.00	7.35	8.75	2.

Kind of Furnace.	Approximate estimate mean temperature of furnace gas, Fahrenheit.		Equivalent evaporation per 212° F. per lb., including water in fuel.	Equivalent evaporation from 212° F. per lb., excluding water in fuel.	Relative efficiency excluding water in fuel—Crockett = 1.	Relative efficiency excluding water in fuel—wood=1.	Relative efficiency including in account water in fuel—Crockett = 1.	Relative efficiency including water in fuel—wood=1.	Absolute efficiencies, water in the fuel.			
									excluded		included, dry wood as standard	
	Furnace	Flue.							Standard of 6385 units.	Dry wood of 6480 units.	of 6385 units.	of 6480 units.
Thompson.....	2118°	544°	5.68	4.24	1.33	0.90	1.29	1.21	0.64	0.63	0.85	0.86
Crockett.....	919°	452°	4.41	3.19	1.00	0.68	1.00	0.94	0.48	0.47	0.66	0.67
Ordinary steam boiler.....	.....	.....	4.69	4.69	1.47	1.00	1.06	1.00	0.70	0.70	0.70	0.70
Ordinary steam boiler.....	2400°	700°	10.50	10.50	3.28	2.23	2.37	2.23	0.70	0.70	0.70	0.70

this effect. The nearer surface is inefficient, and the most distant portions of the heating surface are therefore proportionally much more efficient than in the preceding case.

Extension of surface is, however, precluded by the fact that the temperature of escaping gases would fall still further below that required for effective draught, and, as already indicated, were it possible to operate the furnace at all, this temperature would become one-half the temperature of the boiler, were so much heat abstracted as to give an efficiency of fuel equal to that obtained in the other furnace. Heat would then pass from the boiler to the gas at those portions of its surface farthest from the fire, and the draught could only be maintained by means of special "blowing" apparatus. This is another fact illustrating the importance of high temperature of furnace in the attainment of high furnace efficiency.

64. The preceding table presents the results of the above investigation in a concise form, in which it may be found very useful for reference.

65. Both of these furnaces were introduced about twenty years ago, and the first is in somewhat extensive use. No experimental determination of their actual relative efficiencies has ever been made before, so far as the writer is aware, which has enabled their theory to be worked out. The determination of their theory, as here given, has greatly interested him, and will, perhaps, prove as interesting to the profession. It may be found of value, in view of the many important applications which are daily being made of the various kinds of wet fuel.

The temperatures, as given, may be somewhat below actual temperatures where they are determined from the composition of fuel, as the calculations are made on the assumption that all vapors issuing from the fuel are raised to the mean temperature estimated. The real fact is, that they are expelled while the temperature of issuing gases is reduced by their presence, and they therefore do not abstract as much heat as is debited to them in the calculation.

During those intervals of time which elapse between the drying of one charge and the introduction of the next, the temperature of furnace rises to that due to the combustion of dry fuel. The result, as given, are probably, however, practically and sufficiently correct.

Mr. Richard H. Buel—In tests such as are here described the principal points to be determined by experiment are as follows :—

1st. The pressure of steam.

2d. The weight of water evaporated.

3d. The quality of the steam.

4th. The temperature of the products of combustion in the chimney.

5th. The temperature of the feed water entering the boiler.

6th. The weight of fuel supplied to the furnace.

7th. The amount of effective combustible matter in the fuel.

1st. It is not stated in this paper whether or not any tests were made to determine the accuracy of the steam gauges. If this were not done, the record of pressures is very unreliable. According to the reports of the Hartford Steam Boiler Insurance Co., defective steam gauges are only too common. The temperature and heat of the steam are used in the principal calculations of the paper, rendering it of the utmost importance that the pressures should be accurately recorded.

2d. In the experiment on the Thompson furnace the volume of feed water was obtained, not by continuous observations, but by a series of tests at intervals ; how many is not stated. Engineers who have tried the experiment of making a pump give a constant supply—as, for instance, maintaining a steady head in a tank from which the water is constantly flowing—have found that it was necessary to regulate the opening of the steam-valve of the pump continually as the pressure in the boiler changed, or the pump was lubricated. It appears from the calculations in this paper that the steam pressure did vary somewhat, so that it is doubtful whether the pump delivered water continuously at the rate of 90 cubic feet an hour. Again, the weight of the feed water, which had a temperature of 205° Fahr., is assumed to have been 62.5 pounds per cubic foot, which is greater than the weight of distilled water at the temperature of maximum density. Judging from the experiments of Sorby, Kopp, Matthiessen and Rosetti, it is probable that the weight of the water at this temperature was about 60 pounds per cubic foot. Certainly if it were not weighed on the occasion of the test, there is a chance of inaccuracy in the record. The mode adopted with the Crossett furnace for measuring the feed water is far more certain.

3d. The method employed to test the quality of the steam is one of the most elegant and precise that can be employed without the use of costly apparatus. It seems to have been somewhat defective in the present case on account of the small number of observations.

The *per cent.* of priming affects the performance of a boiler very seriously, and it is believed that the true mean cannot be assured from three or four observations which differed from each other so widely, as stated in this paper.

4th. The determination of the temperature of the flue seems open to the same objection. Only two observations are recorded, giving results which vary more than 40° from each other, so that it is questionable whether the arithmetical mean of these two quantities represents the mean temperature of the chimney during the trial of 13 hours.

5th. The temperature of the feed on entering the boiler, being merely an estimate, is, of course, somewhat uncertain. It is generally considered by engineers that a precise test requires the actual observation of the necessary data, and that where estimates are the only available data, they detract considerably from the value of the final results.

6th. The weight of the fuel supplied to the Crockett furnace was not determined by experiment, as in the case of that used in the Thompson furnace, and as the fuels used in the two cases were said to vary somewhat in quality, and to contain different percentages of water, it is doubtful whether the estimated weight is correct.

In calculating the efficiency of each furnace, it is assumed that there has been no loss of water in transferring the fuel from the leach to the furnace, while in the calculation for furnace temperature, the result of which is compared with the former, it is estimated that some of the water is lost in the transfer.

7th. It may doubtless seem questionable to many whether such small samples as could be placed in fruit jars can be assumed with certainty to represent the average quality of the whole mass of the fuel, since a few drops of water, more or less, which might accidentally be introduced or excluded would change the percentage of moisture to a serious extent.

It will probably be admitted by most engineers that one of the most admirable reports in print is that on the Lowell pumping engine, in which the observed data and the results alone are given, the former being so complete that any one can retrace the steps by which the conclusions have been reached, and so test their correctness by independent calculations. It is believed that this furnishes a standard by which to measure the value of other reports; in an attempt to reproduce the results obtained in the paper under consideration, the present writer first had his attention called to the points which have just been enumerated.

DISCUSSIONS.

ON UPRIGHT ARCHED BRIDGES.*

Mr. James B. Eads.—In reviewing the criticisms upon the paper before the Society, the author will first notice such as have a general bearing upon the subject of bridge building, and afterwards those which relate more especially to the merits of the proposed system.

The principle that, the "engineering is best which most fully answers its purposes at the least cost," as expressed by Mr. Welch, is fully recognized by the author, and it is upon this axiom he bases his belief that for spans of considerable length, the system under discussion must ultimately supersede every other one now in use. It may, however, be inferred from Mr. Macdonald's remarks that the author is more ready to preach than he was to practice in the St. Louis bridge the excellent axiom of Mr. Welch; therefore he will briefly refer to them, with the prefatory remark, that whenever it is necessary to test the engineering merits of the design of that bridge by the axiom stated, he will be quite willing to abide the issue.

Referring to the accurate workmanship and expensive material in the arches of the St. Louis bridge, Mr. Macdonald says he doubts if the author, "will again adopt this design for the purpose of saving a few pounds of metal." The inference from this is that; 1st, the proposed system must also in all cases involve the same accuracy and costly material as the St. Louis bridge; 2d, that accuracy and peculiar material enhanced the cost of that work, and 3d, the design for that structure was adopted simply "for the purpose of saving a few pounds of metal."

The first inference will be shown to be unfounded before closing this review, and it will be immediately apparent that the second and third are not justified by the facts.

The ribbed arch system, notably illustrated at the time, in the 350 feet arches at Coblentz, was finally adopted as that on which the design should be made, because the entire bridge could be built in conformity to the law, and with the requisite strength and capacity for less money on that system than on any other, of which the author and his assistants had knowledge; and it is scarcely saying too much, to assert that

* Continued from page 288.

this fact was arrived at, through an amount of careful comparative examination of various lengths of spans, and corresponding foundations, for both trusses and arches, and after such a series of laborious investigations and study as were never before bestowed upon any similar work.

Before adopting the design, satisfactory assurances were obtained from practical steel makers, that the requisite material could be supplied in perfection, and at less cost per pound for the finished steel than double that of wrought-iron; while the maximum compressive strains it would bear with equal safety (27 500 pounds per square inch), were nearly, or quite three times as great as could be imposed on the iron. The contract price of the steel work delivered at St. Louis, excluding erection, was 15 cents per pound, and the amount of machine and hand work put upon it, after it came from the mills, was less in proportion to the mill prices than that usually put on wrought-iron bridge work. The mill price was 11¼ cents, delivered in Pittsburgh. Allowing ¼ cent for transportation to St. Louis, there were left 3¼ cents, or about 28 per cent. on the mill price for finishing. The tubes and couplings involved the most accurate part of the work, while they constituted by far the largest portion. The average weight of a tube section and coupling being about 5 000 pounds, it will be seen that the contractor received about $162.50 for finishing each; or about $170 000 for all the tubes and couplings. Although unforeseen difficulties were met, as is usual in producing new forms in steel and iron, these were all skillfully surmounted by the contractors without extraordinary exertions, and I am quite confident that both the mill work and the finishing could be duplicated at these prices to-day very readily. If the finished steel had but double the strength, and cost double the price of iron, it would be much more economic in either an arch or truss. To show why it is so, at these relative ratios of price and strength, it is only necessary to refer to the simple fact that the arch (or truss) must support itself before it can support anything else. If the curve of the middle arch of the St. Louis bridge were reduced to about one-quarter of its present height, and were of wrought-iron, the safe limit of strength in the metal would be already reached, and it could not support any part it carries, even of the superstructure, without exceeding that limit; whereas if it were of steel of double the strength per square inch, and but half the sectional area or quantity were used, it would have a surplus of one-half of its strength yet left, and could consequently carry with safety a load equal to its own weight.

By increasing the curvature of the iron arch from this low versed sine (of about 11 feet) it would gradually acquire a surplus strength,

available for supporting the roadway and live load, as the horizontal forces would be lessened by increasing the depth of the arch. But even with the versed sine of the centre span (47.5 feet), supposing the allowable strain on the iron to be 10 000, and on the steel only 20 000 pounds per square inch, the iron would require double the sectional area of the steel, and 16 per cent. added to that increased area, to equal the available strength of the steel. But as the steel really bears 27 500 pounds instead of 20 000 pounds, it would require that the iron should be doubled and 64 per cent. of this doubled weight or section added to it, to possess an available supporting value equal to that of the present steel arches. These arches weigh (exclusive of vertical and horizontal bracing) nearly 4 000 000 pounds. If of iron, their weight must have been doubled and had 64 per cent.* of the doubled quantity added to it. In short, they would have weighed about 13 000 000 instead of 4 000 000 pounds, whence it is evident the author did not adopt the design simply, "for the purpose of saving a few pounds of metal," but to save several million pounds

As the contract price of the steel was 15 cents per pound, or about $600 000,—and as the work, if in iron, would have cost at least 10 cents per pound, or about $1 300 000,—it must be apparent that the steel was more economical. The erection of 6 500 tons of metal would also have cost largely more than that of 2 000 tons. Besides this, the horizontal thrust of each arch would have been increased about 2 000 tons, and this would have involved the necessity of a much greater quantity of masonry, to resist this horizontal force. It will therefore be seen that there were several good reasons for using the more expensive material.

While the author feels sure he could with his present experience materially lessen the cost of the bridge, if it were to be executed again by him, he is quite certain he could not do so by making any important change in the design, if the ribbed arch were used. No ordinary truss system could be so cheap. The only important improvement he can suggest upon that design, is contained in the paper submitted.

If judged by the rule stated by Mr. Macdonald, the design will be fully vindicated. He says, "in most cases the problem of covering a given space is best solved, and with the greatest economy, when the cost of superstructure about equals that of its supports." As originally designed, the substructure (piers and abutments) was estimated at $1 540 080, which was only 879 662 more than the original estimate for the super-

* The above figures are only designed to be approximately correct. A careful investigation, in which all of the elements involved are included, would doubtless modify the percentages named. They are, however, sufficiently correct to show the enormous difference in weight between the requisite quantities of iron and steel.

structure, or 2½ per cent. on the aggregate of both; while the actual cost of the substructure was for

Abutments	$1 000 214.35
Piers	1 073 689.91
Tools and machinery	290 548.00
Total	$2 364 452.26
Superstructure, (excluding approaches)	2 234 655.42
Difference	$129 796.84

The original estimates were increased by some alterations in the design; which consisted in sinking the east abutment pier to the bed-rock 60 or 70 feet deeper than was first intended, and in using a large quantity of granite ashler; also in doing much of the work at night and on Sundays, under the delusive hope that the superstructure would be completed in the contract time. The cost of superstructure was chiefly increased by widening the bridge 4 feet, placing an iron wind truss under the upper roadway, and making some other modifications in the design; and also largely by extra payments to induce the contractors to hasten its completion. Still with all these, the difference in cost of sub and superstructure is only about 3 per cent. on their aggregate cost.

This evidence is not advanced because the author has faith in Mr. Macdonald's rule; but because it is testimony, which to him may be interesting, if not convincing.

The addition of the general items of engineering and contingent account proportionately charged ($400 000), will make the actual cash cost of the bridge, excluding approaches, as above stated, $4 999 107.68.

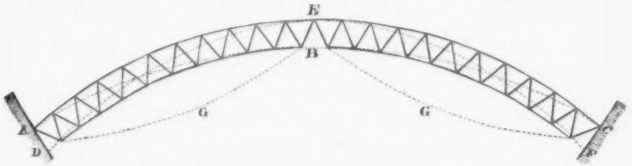
The total outlay as given by the auditor for everything in any manner directly connected with the construction of the work (excluding the tunnel and land damages but) including salaries, rents, engineering, steam and street railways, gas and water-pipes; all of its approaches; losses by failure of contractors; extra compensation and bonuses to expedite the work, &c., &c., amounts to $6 680 331.47.

The additional cost of the bridge aggregating nearly $6 000 000 more, consists almost wholly in items of discount, interest and commissions on bonds; charters, legal expenses and similar expenditures, including $1 086 000 for real estate and right of way. These have, however, no bearing upon a discussion of the merits of the design as an engineering question. In judging of its economic merits, engineers should not lose sight of the fact that a broad and unobstructed street with its sidewalks, and roadway for four lines of wheeled vehicles, in addition to two lines of steam railways, is carried over exceptionally long openings, and

upon unusually deep foundations, the maximum moving load being equal to that of three ordinary railway bridges.

Mr. Shreve says: "the paper before the Society appears to be an acknowledgment of the 'disadvantages of the system adopted for the superstructure of the St. Louis bridge.'" This is an error, and misled by it, he falls into several others, in criticising the St. Louis bridge. He says: "suppose the crown to rise, the lower tube if relieved of compression at the abutments, would not be strained at any point. Bracing beneath a parabolic arch uniformly loaded, can receive no strain and is useless, and if the curve of the tube considered is not a parabola, it is so near it that the strains, which are transferred by the braces to the lower tube are too small to be noticed. The conclusion is unavoidable that the upper tube at times must bear the whole load, and even more, for the tubes 'are rigidly held to the abutments by anchor bolts,' and a large extra amount of strain is in this manner brought upon the loaded tube." When cold causes the crown of the arch to be depressed, he declares that then "the lower tube must just as surely take the whole load, and the upper one be useless."

Mr. Shreve suggests a modification of the ribbed arch by which this supposed alternate imposition of the entire load upon the upper and lower members can be avoided. As the diagram with which he illustrates this furnishes a very convenient means of demonstrating his errors respecting the St. Louis bridge, it is reproduced with modifications.



This diagram differs from the St. Louis arches in having a section of the lower tube omitted at the centre and at each abutment, and in having the upper tube jointed at the centre and at each abutment.

Increase of temperature will cause the arch to rise. This will shorten the central space in the lower tube. If the omitted central section were in place when the arch begins to rise, it would evidently be compressed; and if the temperature were sufficient, the entire compressive strain would certainly be taken off the central joint in the upper tube, and be transferred to the restored section in the lower one. No compression could come on the lower tube at the abutments if the omitted sections there were restored, as the rising of the arch would increase those spaces. The whole load would, therefore, be still sustained at the

abutments by the upper tube, but *by the lower one at the centre.* Mr. Shreve thus falls into another error in supposing that "the lower tube if relieved of compression at the abutments, would not be strained at any point."

If the omitted sections at the abutments were restored and the arch were depressed by cold, the compression, which in the preceding case existed in the lower tube at the centre, would be transferred (in proportion as the depression increased) to the central joint in the upper tube; while the compression in the upper tube at the abutments would be taken off, and be transferred to the lower one at these points. Mr. Shreve is, therefore, a third time in error in supposing that when the crown of the arch falls "the lower tube must just as surely take the whole load and the upper one be useless," because the whole load is then sustained at the centre of the arch *by the upper tube.*

The centres of pressure in these two opposite cases would alternately follow the dotted lines *A B C*, and *D E F*. Greater extremes of temperature would throw these lines at the centre, above and below (or outside of) the arch, and *tension* would thus be created in the middle of that tube which would be in compression at the abutments.* Where these lines of pressure intersect each other in the haunches of the arch, the load must always be borne in nearly equal portions by each tube. In these parts of the arch it can never be imposed upon one tube alone. Hence Mr. Shreve falls into a fourth error when he declares that "each tube must contain a quantity of material sufficient to resist the strains caused by a full load."

Extremes of temperature are thus shown to cause in the tubes great alternations of pressure only in the crown and near the abutments; and these extra pressures can only exist at the same time in the crown of one tube and the abutments of the other, and *vice versa.* They result from temperature in the metallic ribbed arch, whether its curve be parabolic, circular or elliptic, and whether uniformly loaded or unloaded; and they can only be transferred from the ends of one tube to the crown of the other *by the bracing.* Hence Mr. Shreve makes a fifth mistake when he asserts that "bracing beneath a parabolic arch uniformly loaded, can receive no strain and is useless."

There is no difficulty in determining the location of the centre of pressure in any part of the ribbed arch under the different conditions of load and temperature, when the moduli of elasticity and of expansion by heat are known; and hence there need be no uncertainty in proportioning

* This tension can never occur in the St. Louis bridge, owing to the initial stress caused by the weight of the superstructure.

each tube to bear its part of the maximum pressure, even if its ends be bolted to the abutments. Mr. Shreve, however, thinks there must be, and declares as an axiom that "a plan in which such ambiguity of strains obtains is sufficiently defective to be rejected without further examination." His plan of investigation seems, however, to have escaped the crucial test of his axiom, for its results are not without considerable "ambiguity," inasmuch as they lead him to commit a sixth error in declaring that "the strains which are transferred by the braces to the lower tube are too small to be noticed," when in reality, with a full load and maximum temperature, these strains will equal about 5 000 tons in any one of the arches he has criticised.

Mr. Shreve makes a seventh mistake, in assuming that 25 per cent. should be added to the material of the tubes over what would be required if the same material were used in tension. The ratio of length to diameter in compression members, is too important an element to be thus lightly considered. The character of the material must also be taken into account. A prudent engineer would not trust cast-iron in tension with half the strain he would allow in compression, provided the material were in the form of a tube 6 or 8 diameters in length. The cast-steel in the tubes of the St. Louis bridge will bear safely much more in compression in its present form than it could possibly do in tension. Hence the extra 25 per cent. assumed by Mr. Shreve is wholly erroneous.

The dotted lines G, G, placed below the lower segments in the diagram, by which the curves of these segments are reversed, will at once show that greater resistance under partial loading will result if the depth of bracing be increased by thus reversing the curvature of these members. Thus modified they will closely resemble the counter-arches suggested by the author, and exact calculation will demonstrate that in proportion as they approximate them in form, the economy of the structure will be increased. Referring to the jointed halves of his diagram, Mr. Shreve says; "that the greatest tension in the lower arc of either segment is when that segment alone is fully loaded, is self-evident." This constitutes Mr. Shreve's eighth mistake. Exact calculations show that this is neither true of the rigid arch, like those at St. Louis, nor in one with three joints like Mr. Shreve's illustration. In both cases greater tension will occur in the lower members when the maximum load per foot has advanced so as to cover either three or five-eighths of the entire span. This fact is shown graphically in Fig. 14, accompanying the paper.

Mr. Shreve attempts to demonstrate that 57 per cent. of the material used in the St. Louis arches would have accomplished equal results with ordinary horizontal trusses; whereas it is really beyond the

power of any one to show that thrice 57 per cent. of it, could accomplish any such thing.* He closes his criticism of the St. Louis bridge with the remark that "engineers are likely to be satisfied, for the present generation at least, with what has already been done in the construction of steel arches." This corollary is certainly sustained by his method of expounding the principles involved in them.

Maj. Pope makes an admirable presentation of the apparent theoretical and practical advantages of the suspended arch over the upright one. But in doing this, he makes out a stronger case against the horizontal truss system which he advocates, than against the one under discussion, as we shall presently see. In his comparison, between the two systems of arch construction, it must be manifest that every objection urged against the arch applies equally to the bow-string girder; which is simply an arch provided with a chord instead of abutments, with which to resist the horizontal forces. Whether these forces be resisted by the chord or by the abutments, the necessity of maintaining the integrity of the plane of the arch is equally and absolutely imperative. If we reverse the arch in the bow-string girder, we reverse the strains, and have to all effects and purposes, a suspended arch, as much so as if back chains and anchorages were substituted for the compressive chord. In one case the centre of gravity is above the points of support and in the other below. As it is necessary to brace all bridges against lateral forces, so as to make them practically immovable, no more bracing can be needed for the bow-string girder whether the centre of gravity be vertically above the points of support or vertically below them. The element of gravity cannot come into play to increase the strains on the bracing in the one case, nor to decrease them in the other, so long as the plane of the arch remains vertical and the centre of gravity is in that plane. If we are dealing with structures that are to be permitted to sway from side to side, the case will be quite different. In such event the suspended arch, and trusses of the Fink type, may do with less lateral bracing than the

* I will state for the benefit of those who desire to investigate this subject, that one arched rib of the centre span at St. Louis (exclusive of lateral bracing) weighs 490 342 pounds. Of this, 328 286 pounds are in the tubes (including bands and couplings). The tubes are all of cast-steel, tested to 55 000 pounds per square inch without permanent set, and the highest maximum strain they must bear will not exceed 27 500 pounds. The remainder of the rib is of wrought-iron, the allowable strains on which are 10 000 pounds per square inch. Four of these ribs constitute the span. The total dead load of the span is 8 000 pounds, and maximum live load, 6 400 pounds, per linear foot. The clear span is 520 feet, and the versed sine 47.5 feet. The upper roadway is an unobstructed street, 54 feet in extreme width, with double line of railway below. Allowable tensile strains in wrought-iron carrying the railways, 5 000, in other wrought-iron 10 000, and in steel 20 000 pounds, per square inch. Any other data that may be desired will be cheerfully given.

upright arch or upright truss, but not otherwise. This fact must be evident on reflection. It is only when the bridge is permitted to sway to the one side or the other that the gravity of the structure can be made available to lessen the strains on the lateral bracing. Whenever we assume that no swaying shall occur, we must assume that the element of gravity cannot enter into the calculations which determine the strength of the lateral bracing.

It being a fundamental condition that oscillations shall be reduced practically to zero in first-class bridges, the advantages of gravity claimed by Maj. Pope for the suspension bridge falls likewise to zero. It is only during erection that the effects of gravity involve greater care, and in all cases, probably somewhat greater expense, in the upright arch or truss, than in the suspension bridge. But it is questionable whether we can safely trust lighter lateral bracing even in a structure that is permitted to oscillate; for the momentum incident to the vibration may cause the destruction of the bracing before the opposing element of gravity is brought sufficiently into play to save it.

The bow-string girder being simply an arch, with a chord instead of abutments, it is evident that to insure equal perfection, security and stability for its compressive member, there must be precisely as accurate workmanship, equal care in proportioning the length and diameter of its individual parts, and the same amount of lateral bracing as though its horizontal forces were resisted by abutments. The question then naturally suggests itself, if these are fatal objections in a bow-string girder, wherein does the horizontal upright truss differ from it so much, that its compressive member can have less accurate workmanship, less regard for the relation of length to diameter in fixing the sizes of its various parts, and less lateral bracing to retain it in its upright position.

Maj. Pope says: "Each rib is by itself utterly helpless, even to carry its own weight, and must, therefore, be connected with other ribs by an elaborate and expensive system of lateral bracing. Its construction must be tubular, or rectangular, or of some similar form suitable to resist flexure, and all such forms are expensive. It abounds in joints, each one of which must be carefully and perfectly fitted. The lateral bracing must be so complete as to thoroughly secure the various ribs at intervals of not greater than ten or twelve diameters. To erect it in place is a difficult and dangerous task. The centre of gravity is high relatively to its base, and, consequently, the forces of nature act against rather than with it." The author is unable to discover in this attempt to point out the disadvantages of the upright arch, a single item which does not as well apply against the system Maj. Pope is defending.

Engineers do not need to learn that in resisting equal compressive strains in any truss system, it is necessary to observe the same laws in proportioning the sizes of the members and their sectional forms, and that there is the same need of accuracy in adjusting their bearing surfaces, as if they were to be used in an arch. In every horizontal system of trussing, where the points of support are beneath the centre of gravity, it is just as imperative that the vertical plane in which the rib stands, shall be preserved, as if the rib were an arch, and that it is as helpless until braced to its mate. To retain the arched rib, or the horizontal one in place, the resistance to lateral motion must be found at the points of support below the centre of gravity, and Messrs. Pope and Clarke, who question the economy of the lateral bracing required in the proposed system, surely will not challenge these propositions:

1st. With a given base, the greater the *height* at which the compression member of a truss is held, the greater will be the bracing required;

2d. The greater is the *length* of the member held at a given height, the greater will be the bracing required; and

3d. The greater is the *weight* of the member held at a given height, the greater will be the bracing required;—in each, all other conditions being equal.

Now, it is incumbent on those who assume that the upright arch system proposed cannot be so cheaply braced laterally, to show what conditions exist in the truss system which overbalance the advantages the arch possesses by virtue of these axioms;—

1st. The truss must have equal height with the arch for economy.

2d. The *length* of the member held at the greatest height in the truss is much more than in the arch.

3d. The *weight* of the portion of the member held at the greatest height in the truss is much greater than in the arch, for three reasons—first, that portion is much longer; second, the section of the truss is greatest in the middle, while that of the arch is least in that part; third, the weight of the truss being greater than the arch with its chord in the proposed system, the compression member of the truss must bear the weight of a heavier chord, and hence must have greater section at its centre on that account. That the wind bracing of the truss is really more expensive must, therefore, be evident.

When the fact is recognized that, in metal arches, the lengths of the individual members composing the arch should be straight, and not curved from joint to joint, these members can be of any form of section used in any of the truss systems, and they may be of cast or wrought-

iron, riveted plate, angle or channel bar, or of any form of iron or steel suitable in any truss of equal dimensions, the objections urged by Messrs. Macdonald, Pope, Clarke and Nickerson will apply with equal force against every truss system in existence. Whether the members are butted against each other at the joints, secured by couplings, riveted together or held by any other proper method, there is no more excuse for permitting inaccurate workmanship in the truss than in the arch, nor can it possibly be shown that there is less danger from inaccurate joints in the truss than in the arch. In the ribbed arches at St. Louis the case was unusual, because two long lines of tubes were extended from skew-back to skew-back, parallel to each other, only 12 feet from centre to centre, to form a rib, and the individual members were only about 12 feet each in length. It will be apparent that if a straight truss, 520 feet long and 12 feet deep, were required, that equal accuracy would be needed in each piece, and that if the truss were four times the depth, and each piece two or three times as long, less accuracy would be required. Still nothing more accurate than $\frac{1}{100}$-inch was demanded in fixing the lengths of the members, and this is but little more than the $\frac{1}{64}$-inch admitted by Mr. Nickerson to be easily attainable. The ribbed arch, owing to its shallow depth of trussing, does involve more accuracy than deep horizontal systems; but the system proposed is not the ribbed arch. In all bridge factories, tools are provided by which the ends of compression members are cut to any required angle, with accuracy and facility, so that the different members in the arch, each being, as it should be, straight, the manufacture per pound for the one kind of structure should be no more costly than the other. The difference per pound, on the system proposed will be unobjectionable.

In all truss bridges where the points of support are below the centre of gravity, the lateral strains in the top members are transmitted either wholly through the end struts by virtue of their stiffness, to the points of support at the ends of the truss; or else partly by these struts and partly by the intermediate ones, down to the floor-system, and thence by that system to the ends of the truss. This being the fact, and it being true that the compression members of the arch at the ends can be straight and stiff, just as well as the end struts of the truss, and that they incline at an angle by which the distance to the floor-beams is much less than in the end struts of the truss, and therefore more easily braced from the floor system, and that two-thirds or two-fourths of the length of the arch is much nearer the floor system than is the compression member of the truss; the author is at a loss to comprehend how any

engineer, who fully understands the system of lateral bracing in the drawings, or the facility which the greater proximity of the arch to the floor-system gives for any other reliable and economic arrangement of lateral bracing, can question the ease and cheapness with which this bracing can be provided.

Maj. Pope and Mr. Clarke both evidently misapprehend this matter, which is partly owing probably to an inaccuracy in one of the drawings. The sectional views of the bridges, however, show that there is no necessity for the roadways being obstructed by the bracing. Maj. Pope is mistaken in supposing that the chords are simply suspended from the arches. They are secured to the arches—in addition to the suspension rods—by vertical struts at the centre of the arch and midway between the centre and the ends of the arch, as will be seen on the drawings. Between these points their weight and that of the roadway should prevent them from rising, as they are perfectly straight, and are held from descending at the intermediate points by the suspension rods. Introducing more struts would not add to the cost, as they act as suspenders. The floor system, however, being thoroughly held vertically, and strongly braced laterally, as is shown by the drawings and tables, would not be quite "as limber as a whip-lash." The compressive strains to be borne by the chords are as safely provided for as in the top members of any railway trusses that are built. In both designs the lateral floor bracing is very thorough and ample.

Mr. Clarke assumes that a platform merely suspended from an arch, must sway from side to side under a passing load. He says: "horizontal bracing alone will not prevent lateral motion, for the lower chord being in tension will vibrate like a stretched string." These views seem strange to the author, inasmuch as they are advanced in defending a system of bridge-building in which on every through bridge, the platform carrying the railway is also suspended from its compression member, and has its chords in tension. Mr. Clarke says; "it is necessary to connect the floor by rigid angle bracing with the upper chord, which being in compression never leaves a straight line." The author is at a loss to understand by what law it is assumed that in tension a long member is more likely to leave a straight line, than when in compression. It would follow by this reasoning, that if each were released from its lateral system of bracing, the member in compression would be the last to leave the straight line, whereas the converse is true. Mr. Clarke has here evidently failed to make himself clearly understood. The fact is, the floor system in the through truss

bridge has its chords in tension, while in the Fink truss it has its chords in compression, and the wind truss of the St. Louis bridge, under the upper roadway, has its chords in neither tension or compression. This shows that in either condition the chords are available for bracing. Certainly, the simplest way to secure a platform against lateral movement, is to extend a system of lateral bracing in a horizontal plane, directly between the two opposite points of support, and at these points to secure the system to the chief supporting members of the structure, and thus make its gravity an element of stability for the intermediate roadway. Mr. Clarke's argument would lead one to suppose that it were better to throw up 4 posts 18 or 20 feet above this platform, and then having these strongly braced against lateral motion, extend a horizontal system of bracing between them, and from this stilted system at intermediate points, send down struts to the platform below and thus avoid the simple and direct horizontal bracing between the points of support; or else assist the latter to do what it could much more economically do without the help of its lofty coadjutor. Mr. Clarke adds: "if this were done in the bridge proposed by the writer," (that is, 'connect the floor by rigid angle bracing with the upper chords') some of the economy of material claimed for it would disappear." This is certainly very true, but to the author it would seem as unnecessary as to hold the ground floor of a rope-walk by means of rigid angle bracing with the roof above it. The structure which is raised above the roadway, whether it be a truss or an arch, is put there solely to secure the roadway against vertical movement. Lateral bracing is absolutely necessary to preserve its position against the effect of wind and vibration. When this is provided, every additional lateral strain that is thrown upon it by the roadway beneath, must involve the use of more material to transmit that strain up to the top chord, and through it to the end posts, and then down again through them to the points of support, than would be required if that strain were transmitted to those points in a direct horizontal plane. This must be so evident to the distinguished engineers (Messrs Pope and Clarke), who have discussed the question of the lateral stability of the proposed system, that further argument is unnecessary.

As nothing can be more evident than that the horizontal plane between the points of support is the most economic location for lateral bracing, it follows that by rigid connections from bracing located in this plane with the structure above it, any lateral forces affecting the stability of the members above can be more economically brought down to this system, and transmitted through it to the points of support, than is pos-

sible by reversing this order of bracing. It follows, too, that the less distant the upper member is from the lower system of bracing, the more directly and economically can the strain be brought down to it.

The author claims that the economy and facility with which lateral support can be imparted to the arch through the wind bracing of the roadway are advantages possessed by this system over that of the horizontal truss systems. Maj. Pope says, however, that where the lateral floor bracing is weakest in this system is just where it needs the greatest strength, if it be required to support the ends of the arches where the bracing between them is intermitted on account of head room for the locomotive. Is not he forgetting the difference between the chords and the system of vertical arch bracing proposed, and that of the ordinary truss systems? The horizontal forces of the arch are all, by its peculiar bracing, thrown directly on to the chords at the ends, consequently these strains require the chords to be of uniform section throughout, and for this reason they are not like ordinary truss chords, weaker at the ends of the truss, whilst the lateral bracing between the chords in this, like any other truss, is not the weakest at the ends, but the strongest there.

Mr. Whipple's theoretical comparison of the economy of the lever-disks and chord arrangement suggested, seems to the author not exactly fair, inasmuch as he bases his conclusions on the fact that the chord of a bow-string girder braced between chord and arch by uprights and diagonals resists but 39 per cent. of the strain upon the whole material of the truss. From this basis he proceeds to estimate the saving by the chord system proposed, which is incorrect, for the reason that the arch proposed is braced in a much more economic manner against unequal loads, than in the ordinary bow-string girder, and this enables the chord to be lighter, which in turn diminishes the weight of the arch. Again, the chord in the proposed system resists *all* of the horizontal forces of the arch, and therefore it constitutes much more than 39 per cent. of the resisting material, if the lever-disk system is not used. But if it be used, the chord will receive but one-half of the entire thrust of the arch, and thus when it saves half of the weight of the chord, there is an additional saving in the arch itself by being relieved of this much dead weight. It is evident, then, that in such a comparison, the system should first, receive credit for a reduction in the weight of the arch itself, due to a more economic bracing against unequal loading, and second, for the additional saving of material in the arch resulting from being relieved of half the weight of the chord, both of which seem to have been overlooked by Mr. Whipple.

In this connection the author will state, that detail plans for a bow-string girder on the system proposed, 517 feet between the end pins of the chord and with one-eighth versed sine, were carefully prepared under the direction of Col. Flad, to determine a wager that this truss would weigh less than *two-thirds* of the weight of a horizontal truss to carry the same load (2 500 pounds per linear foot) over the same length of span, using the same kind of material and with equal safety; and to be one of a design used in several of the longest span bridges in this country. The weights, strains, &c., of the horizontal truss were to have been supplied 12 or 15 months ago, to determine this wager, but this has not yet been done. This bow-string girder, including everything, weighs 3 100 pounds per linear foot; the arch is of steel, to bear 20 000 pounds per square inch; all the other parts are of wrought-iron, including the chords, which take the entire thrust of the arch.

It will certainly be interesting to the profession to know whether the author would win or lose this wager, under the decision of a designated umpire who stands in the front rank of bridge engineers and by whom all questions of safety, details of calculations, and proper proportions of members are to be examined and revised, and the question of total weight decided, in the event of the decision of Col. Flad being challenged. Perhaps the simplest way of determining the question of economy in the proposed system of bracing the arch under partial loads, will be for those who deny its existence to submit to the Society the details and weights of a horizontal truss of 517 feet span on the above data, and show how nearly it approximates to 3 100 pounds per linear foot in weight.

Engineers who object to this system on the ground that "great increase in lengths of span generally seems unnecessary," lose sight of the fact that if there can be great economy attained in long spans by this system, the cheapening of shorter ones by it must be admitted.

With reference to steel in bridges, it would seem that where its advantages are so plainly manifest as they have been shown to be in reviewing Mr. Macdonald's remarks, it is the duty of all engineers to encourage its use in the construction of bridges, roofs, &c., in preference to iron. The author is confident that if such were given, it would not be long before a quality would be supplied at moderate prices which could be easily worked, and be reliable in tension as well as compression for strains three times as great as the wrought-iron usually put into bridges. Steel may be considered as iron more highly refined, hence it would seem that a structure where weight is such an important element of cost, every encouragement should be given to produce the most superior material.

If this were done, the demand would soon induce a competition that would reduce the present cost.

Mr. Clarke makes an objection to the bow-string girder because "practice has shown that the deflections on a bow-string girder are very great at each end, owing to the want of height of the truss at these points." This fact the author is inclined to believe is by no means well established. It may result from defective designs, and may be partly imaginary. The proposed system, however, enables the fault affirmed by Mr. Clarke to be economically overcome in a manner not possible with the ordinary bow-string. In it, owing to the loss of economy in long compression members, the height of the truss is kept down in spans of important lengths, but in the system under discussion a versed sign of one-sixth or one-seventh will be found to be among the most economic proportions for ordinary lengths that can be given, which will greatly increase the depth at the ends. If this deflection exists in ordinary bow-string trusses to the extent claimed, it results more from the manner of trussing with verticals and diagonals between the arch and chord, than from want of depth. The method proposed, distributes in a manner quite different, the strains from unequal loading through the action of the inverted arches, and prevents the objectionable deflection.

ON THE EFFICIENCY OF FURNACES BURNING WET FUEL.†

Mr. Charles E. Emery :—I desire to express my pleasure at the very thorough manner in which the subject has been discussed. The means adopted for obtaining the temperature of the uptake are novel, and in my opinion as well calculated to give accurate results, as by the use of thermometers, pyrometers, &c., of the class ordinarily sold.

Prof. Thurston is, however, mistaken in the supposition, which the words of the paper imply, that a calorimeter was first used for testing steam boilers at the American Institute trials of 1871. The advantages of the system were pointed out by Mr. B. W. Farey, of England, in "Engineering," Sept. 17, 1868,‡ and an account illustrated by drawings, given of practical experiments made with such an apparatus. By his system the boiler and engine were both tested at the same time, the feed-water being measured in tanks before it was pumped into the boiler, and the quantity of water from condenser ascertained by the use of a weir on the edge of a tank provided with baffling screens to regulate the velocity of approach. Indicator diagrams and records of the tempera-

† Continued from page 318. ‡ Vol. VI, page 58.

ture of the water from condenser were taken at the same time. By these means the number of heat units supplied to water in boiler was ascertained; also, the number required per unit of power in the engine.

This method of testing is designated by "Engineering" the "Farey and Donkin" system, and trials made in accordance therewith have, several times during the past few years, been described in the columns of that journal.

Mr. John Pinchbeck, also of England, appears, however, to have used the principle before Mr. Farey gave it such publicity, he having actually weighed the water from condenser in a large tank.*

The value of a calorimeter in determining the evaporative efficiency of different boilers tested in competition will be universally acknowledged. It will be found also very useful in making experiments with engines taking steam under different conditions from the same boiler. In 1869, when in charge of the steam-engine trials at the American Institute Exhibition, I pointed out by computations founded on the terminal pressures from indicator diagrams that the "percentage of moisture present in the steam" was 20.53 for one engine and 9.01 for another, steam being furnished from the same boiler in both cases; such result being partially due to differences in the management of the boiler.†

Shortly after this, Mr. Leicester Allen designed a modification of the calorimeter originally used by Regnault and others in determining the properties of saturated steam, which he proposed to use for ascertaining the quantity of moisture carried over with the steam from boilers in practical use, by blowing into the apparatus small quantities of steam from time to time, and thus ascertaining the quality of steam "by sample," so to speak.

In the boiler trials at the American Institute in 1871, referred to in the paper, Prof. Thurston's apparatus condensed all of the steam, as in the Farey system, but a surface condenser was employed and the condensing water measured with meters.

In ordinary cases where the work is uniform, I think it quite sufficient to test by the calorimeter, portions of the steam supplied by the boiler. In October, 1873, I put in use a simple calorimeter, somewhat like that referred to in the paper, using, however, a wash-tub instead of a barrel, and a coil of iron pipe instead of a hose. A flat piece of wood, used as stirrer, in connection with the spiral shape of the coil, served to

* Engineering, Vol. VIII, page 28.

† Superintendent's Report, Transactions American Institute, 1869–70.

keep the temperature of the water uniform. I find, however, that I used a formula to obtain the results somewhat simpler in form than that given in the paper. As it may be of interest, it is here reproduced, using, as far as possible, the same characters as in the paper:

Let U = total number of heat units imparted to water; t = number of heat units transferred per pound of water added; L = latent heat of steam at observed pressure; W = weight of steam and water added, and x = weight of steam added, then $W - x$ will equal amount of moisture present with the steam. Now the whole of the steam, together with the moisture present in the same (W), transfers to the calorimeter the number of heat units (t) corresponding to the difference in temperature of the original steam and that of the water in calorimeter at the close of the experiment. The steam added (x) transfers also its latent heat (L) to the calorimeter, so

$$W t + L x = U \text{ and } x = \frac{U - W t}{L}.$$

Errata.—On page 254, for the second table read the following:—

Load (allowing 15 Tons for each Tender).			Deflection (inches) at Centre of			
Locomotives.	Weight in Tons.	Placed on Middle of	Rib *A*.	Rib *B*.	Rib *C*.	Rib *D*.
7	334	South Track of Span III	2.48†	1.84†	1.33‡	0.83‡
7	334	" " " " II	2.8†	2.15†	1.37‡	1.1‡
7	350	" " " " I	2.48†	1.8†	1.27‡	0.82‡
14	334	North " " } Span I	3.048	2.802	2.616	3.
	350	South " " }				
14	334	North " " } Span II	3.48	3.44	3.89	3.92
	350	South " " }				
10*	492	" " " " II	2.37	....	....	.....

* Did not cover whole span. † Loaded rib. ‡ Rib partially loaded.

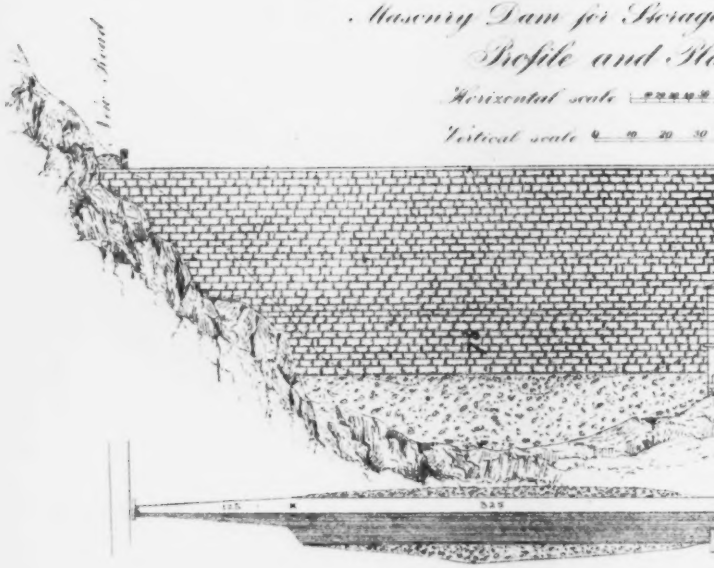
On page 286, fourteenth line, for "212 500," read "215 500"; sixteenth line, for "333 700," read "339 700"; at beginning of seventeenth line, insert "whence 3 397 000 pounds"; and in twenty-fourth line omit "it can be."

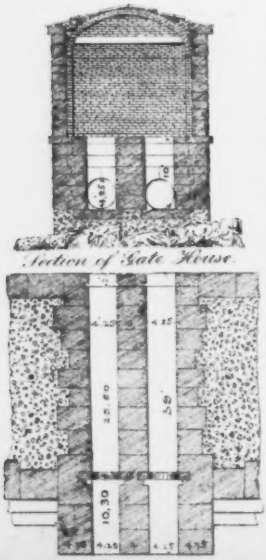
Masonry Dam for Scriag

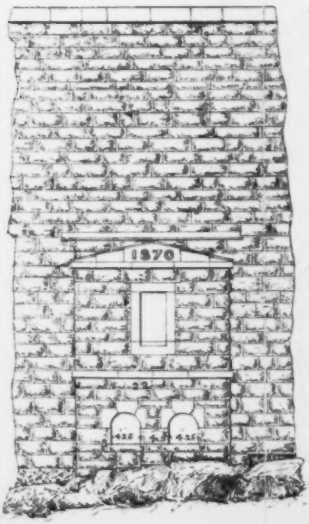
Profile and Pl

Horizontal scale

Vertical scale 0 10 20 30

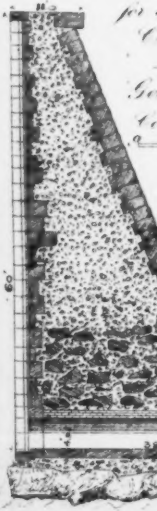






Elevation of

Dam and Gate House



Sect

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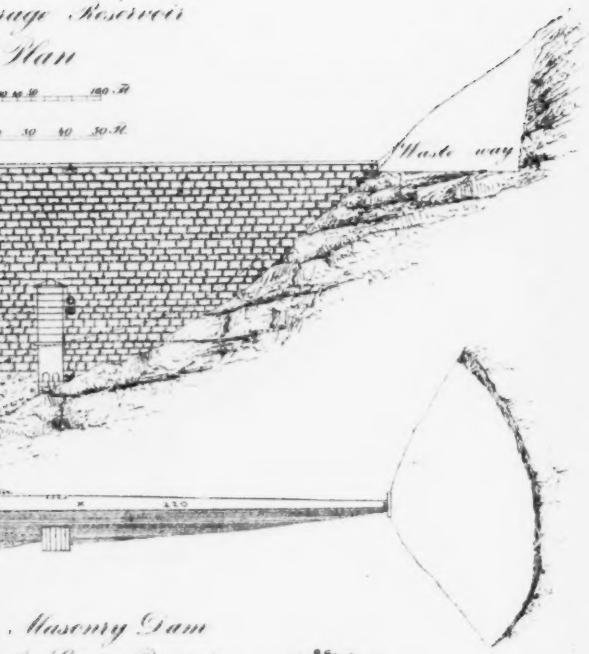


...Masonry Dam

...for Storage Reservoir

Croton Aqueduct

Designed by

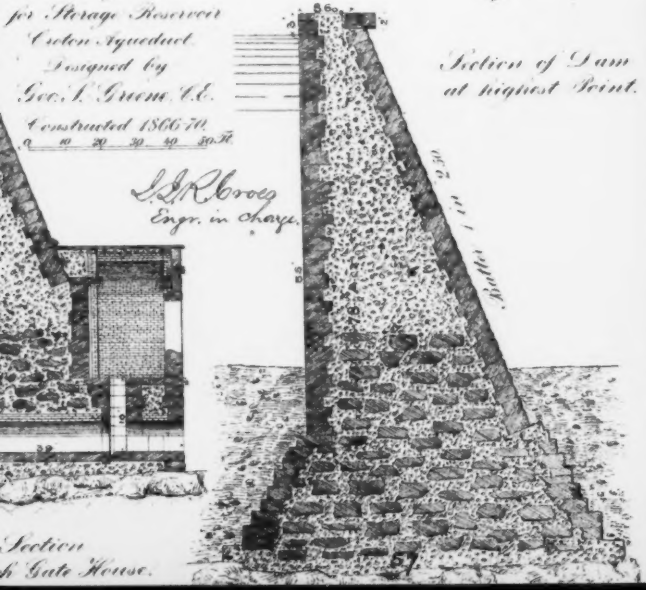
Geo. S. Greene C.E.

Constructed 1866-70.

0 10 20 30 40 50 ft

S. D. Kirkwood

Engr. in charge.



Section of Dam at highest Point.

...Section

...h Gate House.

Fig. 1.

Fig. 3. S. 88.

Road.

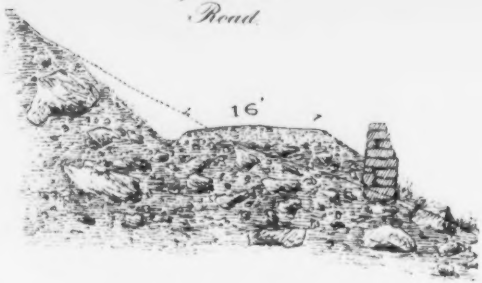
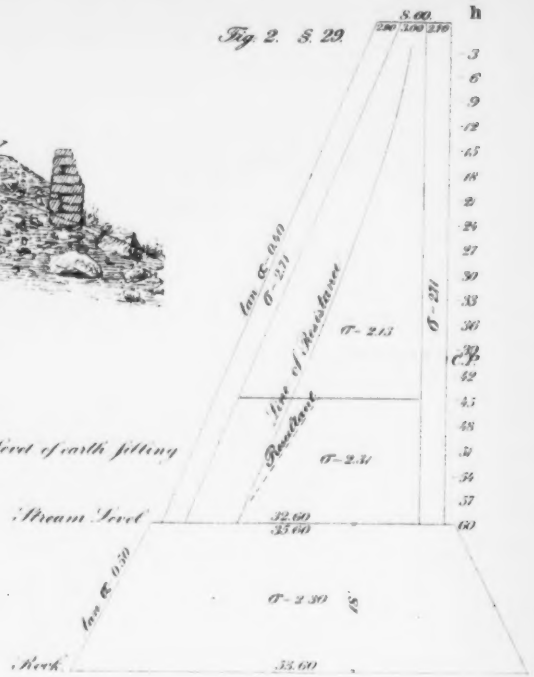


Fig. 4. S. 84.

Fig. 2. S. 29.



A.M. PHOTO-LITHOGRAPHIC Co N.Y. (OSBORNE'S PROCESS)

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CIII.

MEMOIR OF THE CONSTRUCTION OF A MASONRY DAM.

A Paper by J. JAMES R. CROES, C. E., Member of the Society.

FOR WHICH THE NORMAN MEDAL WAS AWARDED

NOVEMBER 4TH, 1874.*

PRELIMINARY.—§ 1. The dam described in the following paper was constructed on a branch of the Croton river, in Putnam county in the State of New York, by the Croton Aqueduct Board, the department of the government of New York city which was charged with the construction and maintenance of works for the supply of water to the city.

§ 2. The object of the dam was the creation of a reservoir for storing water to supply the deficiency in the daily yield of the stream, which existed during the summer months of dry years, the minimum flow being not more than four million cubic feet per day, while the consumption had reached double that amount, and was rapidly increasing.

§ 3. The necessity for artificial storage reservoirs had long been foreseen by Mr. Alfred W. Craven, the chief engineer and a member of the Croton Aqueduct Board since its organization in 1849, and in 1857–58 he had caused a topographical survey of the water-shed of the Croton river to be made, with a view to determine available sites for such reservoirs. The survey showed that fourteen such sites existed; but not until April, 1865, was the requisite authority procured from the Legislature to construct a storage reservoir.

* The Board of Censors in awarding the prize of the Norman Medal of 1874, to this paper on "The Construction of a Masonry Dam," wish it to be understood, that such decision must not be considered as the expression of an opinion on their part, as to the correctness of the views entertained by the author, but that the paper, in the treatment of the subject, serves as a model in their judgment for the class of papers, the production of which, it was the intention of the founder of the premium to encourage.

J. W. ADAMS, Pres. American Society of Civil Engineers,
F. A. P. BARNARD, President Columbia College,
J. G. BARNARD, Colonel of Engrs., Brt. Major-Gen. U. S. A.,
} Board of Censors.

November 4th, 1874.

§ 4. Nearly a year was consumed in careful examination and comparison of sites, and on March 17th, 1866, it was determined to build a reservoir at Boyd's Corners on the west branch of the Croton, 23 miles by the course of the stream from the Croton Dam, where the aqueduct begins which conveys water to the city.

§ 5. The reservoir as constructed here, has a water surface of 279 acres, a maximum depth at the dam of 57 feet, a mean depth of 30 feet, and a capacity of 364,000,000 cubic feet.

§ 6. At the site selected for the dam, the hills enclosing the valley approach each other, leaving at the level of the stream a space of only 200 feet in width, and at 70 feet above the stream, 700 feet. The centre line of the dam is at right angles to the axis of the valley at this point, and bears nearly north-east and south-west.

§ 7. The north-easterly hill-side was covered with large slabs of a very compact and fine and even-grained gneiss, piled one upon another, varying in thickness from 6 to 60 inches. The rift of the stone was at right angles to the bed on which it lay; the slabs were at an angle of 10°, and the general slope of the hill was 16½°. The ledge underlying the slabs was of much the same character, but not quite so fine grained. The general surface was smooth, and sloping at the inclination of the hill. Occasional seams were found to run in nearly horizontally, from 5 to 20 feet, then apparently terminating abruptly. The surface slabs made excellent stone for cutting where they were of sufficient thickness to give the requisite beds. It was found impracticable to quarry out of the ledge, good stone for cutting.

§ 8. At an elevation of 15 feet above the stream, the rock was overlaid with about 10 feet of drift, or compact gravelly earth, containing about ten per cent. of its bulk of large boulders. The flat ground through which the stream flowed was swampy on the surface, underlaid by some blue clay, beneath which was a mass of loose boulders and cobble stone, which was succeeded by coarse gravel and sand which extended to the rock, and through which a considerable amount of water percolated.

§ 9. The south-west hill, rising at an angle of 22°, was composed of compact gravel and boulders, overlying a very fine grained unstratified hard blue rock. The surface of this rock was smooth, but its profile irregular and in benches. It lay at a depth of from 5 to 15 feet below the surface of the ground. In the centre of the valley the two classes of rock seemed fused together, no marked seam separating them. The rock here was 18 feet below the stream level.

§ 10. No dam at this place could be made safe or water-tight, without removing all the natural material and founding the structure on the rock. All the earth within a mile of the dam site contained so large a proportion of boulders and coarse gravel as to render it unfit for water-tight embankment, except at enormous expense for separating and screening out the stone. An earthen embankment would have demanded for safety, a tunnel through the rock of the hill-sides for the discharge-pipe at the stream level, and a rock cut of some 15,000 cubic yards in the hill-side at flow line, for waste water.

§ 11. The stream was liable to sudden and severe freshets which could not be prevented from overflowing the dam during construction. Such a freshet would necessarily destroy a partially completed earth embankment.

§ 12. An abundant quantity of stone of excellent quality lay on the hill-sides within a short distance of the dam site.

§ 13. These considerations led to the decision that the dam should be of masonry. Below the level of the stream, the foundation is composed of concrete in which are imbedded large unwrought stones. Above the stream level, the concrete hearting is faced on both sides with coursed stone with cut beds and joints; for 15 feet above the stream, large unwrought stones are imbedded in the concrete; and above that elevation, the hearting is entirely composed of concrete of small stones.

Form and Stability.—§ 14. The form of a trapezoid with a vertical face opposed to the water, was adopted, as affording the requisite stability with the least amount of material and least difficulties of construction. The form and dimensions of the wall are shown in Fig. 1. The notation used is as follows:

h = height of wall, or distance from top to any bed joint.

a = top width of wall.

α = angle of inclination of face of wall, to the vertical.

b = width of wall at any bed joint at the distance h below the top = $a + h.\ tan.\ \alpha$.

d = horizontal distance of the centre of gravity of the section of the wall, from the vertical face. For a homogeneous wall of this form, $d = \frac{1}{3}\left(a + \frac{b^2}{a+b}\right)$.

P = pressure of water on the vertical face = $\frac{h^2}{2}$ when the weight of a cubic foot of water is taken at unity.

σ = specific gravity of the material of the wall.

W = weight of wall = $\frac{\sigma\ h}{2}\ (a + b)$.

Θ = angle of inclination to the vertical of the resultant of pressures,

$\therefore\ tan.\ \Theta = \frac{P}{W}$.

m = ratio which the distance from the exterior edge of the bed joint to the point where the resultant of pressures intersects that joint, bears to the whole width of the joint or b; $\therefore\ m = 1 - \frac{d + \frac{1}{3}\ h.\ tan.\ \Theta}{b}$. To insure there being no tension on any part of the bed joint, m must not be less than 0.333.* In practice, English engineers are said to adopt as a safe value $m \geq 0.125$, while the French engineers adopt the limit $m \geq 0.2$. In the calculations for this dam, the limit is taken $m \geq 0.25$.

n = ratio which the deviation of the centre of pressure at a bed joint, from the perpendicular let fall from the centre of gravity, bears to the whole base. This ratio is the same as $(q + q')$ used by Rankine. From the figure $n = \frac{h}{3\ b}\ tan.\ \Theta$.

φ = angle at which one stone will slide upon another.

§ 15. In all the calculations for the wall, it is assumed that the water will stand at the level of the top of the wall, although the flow line of the reservoir is 3 feet lower. This assumption simplifies the calculations, and provides against any contingencies which might arise in freshets. It is also assumed that the bed joints are horizontal. The batter of the face wall, and the fact that the joints of the courses there are normal to the face, make the bed joints practically inclined to the horizon, and therefore presenting greater resistance to sliding and overturning, than is assumed in the calculations.

§ 16. For purposes of calculation these values are assumed—specific gravity of the masonry, $\sigma = 2.33$; the limiting angle of sliding of dressed stone upon each other, which is given by various authorities as from 31° to 35° is taken at 32°. $\therefore\ \tan\ \varphi = 0.625$.

§ 17. For economy of material, the wall should approach in section to the wall of uniform stability, which is triangular. In this case, such a wall would have a face batter, of which $\tan\ \alpha = 0.687$.† On the other hand, on account of practical difficulties of construction, the value of α should not exceed the limiting angle of sliding of stones on moist mortar.

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* Van Buren, LVII. Transactions (20).

† Rankine's Applied Mechanics, § 216. Equation (14).

Some experiments give this angle at 22°. The angle adopted for the wall is 21° 49′, or $\tan \alpha = 0.4$.

§ 18. The condition of frictional stability for the wall is $\tan \theta < \tan \varphi$, φ being taken at the limiting angle for dressed uncemented stone; if Θ is made equal to φ for a cemented wall, the adhesive power of the mortar is amply sufficient to produce the required inequality; we have therefore $\tan \Theta = 0.625$.

§ 19. The equation of stability of position (*i.e.*, against overturning) for a wall of this form is*

$$W(q \pm q^1)\ b = \frac{h^3}{6}$$

Substituting the values of the factors as given in § 14, and reducing, we obtain for top width of dam

$$a = \frac{h}{2}\left(\frac{1}{\sigma\ tan.\ \Theta} - tan.\ \alpha\right) \therefore \text{ in this case } a = 0.1433\,h.$$

For different heights of dam, the values of a to fulfill the conditions of stability are therefore:

h.	a.	h.	a.
10	1.43	40	5.73
20	2.86	50	7.16
30	4.30	60	8.60

§ 20. As built, $a = 8.6$ for so much of the dam as lies between the points where the plane of the stream level intersects the slope of the hills. Between those points and the extremities of the dam, as h diminishes rapidly, the same width of dam at top would give an unnecessary excess of material. At the same time, a less value of a than 5 feet is not advisable, that being the thickness of the two facing walls, and the least that is efficient for water-tight work. The top width of the dam is therefore reduced between the points above mentioned and the ends of the dam, by deflecting the line of the vertical face until at the ends of the dam it is 5 feet from the front angle, the line of the front angle being straight, and the sloping face a plane surface throughout.

§ 21. Below the level of the stream, the width of the masonry was increased 1½ feet on each side, and the foundation carried to the rock with a slope on each face of 1 to 2.

§ 22. As originally designed, the waste water from the reservoir when full, was intended to flow over the top of the masonry at the north-east end of the dam, the masonry there being left lower than on the rest of the wall. The overfall was arranged in three sections, one at the extreme

* Rankine's Applied Mechanics. § 216 (2 A).

end cut in the rock, 30 feet long at flow line elevation, the second 9 inches higher, over the masonry and 100 feet long, and the third, 1½ feet above flow line, and 70 feet long. The rock at the base of the face wall for 220 feet at this end to receive the base stones, was cut in steps, level on the face and sloping back at the angle of the wall.

§ 23. For drawing off the water in the reservoir, two water ways, each 4½ feet in diameter, were built through the dam at stream level, as shown in the Plate. Grooves for stop plank were arranged on the up-stream face of the dam, and cast-iron gates with composition facings were intended to be placed in a gate-house fitted for the purpose on the lower side.

§ 24. An earth embankment was built against both sides of the dam to 10 feet above the stream level. To prevent percolation of water through the masonry, it was intended that the joints on the up-stream side of the wall should be raked out, and then thoroughly calked and pointed with cement.*

§ 25. In 1870 when the dam was nearly completed, the control of the work was transferred by the organization of the city government under the charter of April, 1870, to a Department of Public Works, which took the place of the Croton Aqueduct Board.

§ 26. The new authorities made the following changes of plan :

1st. On the water side of the dam an earthen embankment was built, 20 feet wide on top, and with slope of 5 to 2.

2d. At the foot of this slope a tower of masonry was built to above the water level. From the tower, in which are grooves for stop-plank, two lines of 36-inch pipe were laid underneath the embankment, and through the water ways, and carried several hundred feet below the dam. Stop-cocks were placed in a vault built against the upper face of the dam.

3d. An excavation was made in the rock at the north-east end of the dam, about 100 feet in width, for the waste way.

§ 27. By the construction of the earth embankment, the masonry dam is changed into a retaining wall, which is impervious, and sustains a bank of earth exposed to saturation. If there could be any certainty that the embankment was impervious to water, the stability of the wall would not be endangered. But the embankment was built upon the original porous gravel, 20 feet in depth, extending entirely under the

* The dam was designed and the specifications prepared by Gen. George S. Greene, Engineer-in-Charge.

bank to within a few feet of the dam; it was built by contract, and not rolled nor thoroughly rammed, but merely carted over. The material is not such as to "puddle" well. It is therefore extremely improbable that the portion adjoining the dam will be secure from the infiltration of water falling on its surface, or the penetration of the water from below, under 60 feet head in the reservoir.

§ 28. If it is saturated, the pressure against the wall is equivalent to that of a fluid of the same specific gravity as the earth. If this is taken only as 1.60, the value of tan Θ becomes at the bed joint (60 feet below top of dam) = 0.996, and the resultant of pressures strikes the base only 1.1 feet inside the edge, making $m = 0.034$, showing that instead of a dam amply strong to resist any pressure of water, the wall has become a retaining wall so near the limit of stability as to be, if not unsafe, at least very imprudent. Its stability now depends *entirely* upon the adhesion of the mortar, for the embankment is not water-tight, and the face of the dam is constantly wet with water which has percolated the masonry for want of calking to the joints on the upper face.

§ 29. The position of the line of resistance in the dam as built to resist a water-pressure, is shown in Fig. 2. The values of σ. *tan.* Θ and m for every 3 feet in height, are as follows:

h.	Specific Gravity of the Mass. σ	Distance of the centre of Resistance from the outer Edge of Base $m\,b$.	Condition of frictional Stability, *tan.* $\Theta <$ 0.625. Actual Values. *Tan.* Θ.	Condition of Stability of Position $m >$ 0.25. Actual Values. m.	h.	Specific Gravity of the Mass. σ	Distance of the centre of Resistance from the outer Edge of Base $m\,b$.	Condition of frictional Stability, *tan.* $\Theta <$ 0.625. Actual Values. *Tan.* Θ.	Condition of Stability of Position $m >$ 0.25. Actual Values. m.
3	2.483	5.12	0.065	0.523	33	2.343	8.66	0.463	0.398
6	2.461	5.82	0.124	0.529	36	2.334	8.73	0.488	0.380
9	2.442	6.41	0.177	0.526	39	2.328	8.78	0.511	0.363
12	2.418	6.91	0.226	0.516	42	2.321	8.80	0.532	0.347
15	2.410	7.33	0.268	0.503	45	2.314	8.80	0.552	0.331
18	2.397	7.68	0.308	0.486	48	2.322	8.80	0.568	0.316
21	2.384	7.97	0.344	0.469	51	2.328	8.78	0.583	0.300
24	2.372	8.21	0.378	0.451	54	2.333	8.76	0.597	0.291
27	2.362	8.40	0.408	0.433	57	2.337	8.72	0.610	0.278
30	2.352	8.55	0.437	0.415	60	2.340	8.68	0.622	0.266

Pressure per square inch of wall on foundation 38.4 pounds; result-

ant pressure on foundation 45.25 pounds; and pressure of foundation on rock 38 pounds.

PLANS AND PROPOSALS FOR CONSTRUCTION.—§ 30. The law with regard to work undertaken by the Croton Aqueduct Board required it to be let at public competition to the lowest bidder. On August 20, 1866, eight proposals were received and opened. The following were the prices offered:

	No. of Bid	1	2	3	4	5	6	7	8
	Class of Work.				Prices				
A	Earth Excavation below stream, per cubic yard	$1.00	$1.00	2.60	$0.75	$2.00	$1.00	$1.35	$2.00
B	Earth excavation above stream level per cubic yard	0.50	0.25	0.90	0.45	0.75	0.50	35	0.50
C	Rock excavation, per cubic yard	10.00	2.00	3.00	0.50	2.50	5.00	4.60	5.00
D	Dimension cut stone masonry, per cubic yard	30.00	60.00	78.84	27.00	40.00	108.00	66.66	40.00
E	Cut range masonry, per cubic yard	15.00	25.00	21.75	20.00	12.00	27.00	47.00	22.00
F	Rubble range " " "	11.00	14.00	17.00	11.93	9.00	10.00	40.00	18.00
G	Concrete masonry below stream level, per cubic yard	8.00	7.50	6.60	9.93	10.00	10.00	11.45	17.00
H	Concrete masonry above stream level, per cubic yard	6.50	7.00	6.10	7.93	8.00	10.00	11.45	16.00
I	Flagging for gate-house roof, square foot	0.20	1.00	0.60	2.00	0.30	2.00	2.40	1.35
J	Earthen 9-drain pipe, linear foot	1.00	2.00	1.00	5.00	2.50	2.00	2.27	4.00
K	Cutting grooves for gates, per square foot	0.30	1.00	1.27	3.00	1.50	2.00	3.77	0.60
L	Brick masonry in arches, per cubic yard	20.00	20.00	22.00	20.00	20.00	27.00	36.67	28.00
M	Temporary road, per rod	10.00	10.00	5.00	5.00	8.00	10.00	6.37	3.00

§ 31. A comparison of these bids is instructive, as showing the different manner in which contractors interpret plain specifications, and the variety of views entertained as to the proportions of remunerative and unremunerative work in an undertaking of this size.

§ 32. "*A.* For Earth Excavation," including all material except solid rock which required blasting, and boulders of more than one-half a cubic yard content, from the dam site below the level of the stream, and including the cost of putting the same in embankment or spoil bank, and all pumping and bailing for keeping the pit free from water.

Bid No.	1	2	3	4	5	6	7	8
Price, per cubic yard	$1.00	$1.00	$2.60	$0.75	$2.00	$1.00	$1.35	$2.

In these figures is seen the effect of the unknown constant for pumping and bailing. The plan for the work required the stream to be diverted from its channel near the centre of the valley, and a new channel made about 100 feet to the north-east, an embankment 10 feet above the ordinary stream level to be built around the pit on the south-west, and the excavation within this bank to be removed down to the rock. Slopes of 1 to 1 were estimated to the contractor, all caves and slides beyond that to be at his own cost.

The amount of water which might be expected to percolate the gravel was estimated by the various bidders at very different figures, their bids showing that it was considered to be worth from $0.35 to $1.70 per cubic yard of earth to take care of the water. The actual cost of the removal of water from the pits during excavation was about 45 cents per cubic yard of excavation.

§ 33. "*B.* For Earth Excavation" above the level of the stream, including everything as specified under "*A.*"

No. of Bid.....	1	2	3	4	5	6	7	8
Prices, per cubic yard.....	$0.50	$0.25	$0.90	$0.45	$0.75	$0.50	$1.35	$0.50

Of these figures, No. 2 is unaccountable and displays great lack of judgment. The excavation for the dam proper was in shallow pits, on a steep hill-side and of very hard material. It was known to be likely that the estimated quantity would be increased for embankment around the dam, the material for which would require to be selected, and to be hauled not less than 1,200 feet, and well compacted in bank. Instead of following the rule, which almost invariably holds good, that for an item which is likely to be increased, a good paying price should be offered, this bidder reduced his price on this item to less than cost. No. 3 exaggerated the cost of this work as he did of the pumping, and No. 7 evidently did not intend to lose on either "*A*" or "*B.*"

§ 34. "*C.* For Rock Excavation," requiring blasting, including the removal of all boulders of over one-half a cubic yard content.

No. of Bid.....	1	2	3	4	5	6	7	8
Price, per cubic yard.....	$10.00	$2.00	$3.00	$0.50	$2.50	$5.00	$4.60	$5.00

The estimated amount of rock was not large, and No. 1 evidently counted on its increase. The event proved him to be right, and this item alone compensated him for a large proportion of his losses on other items. The excavation contained from 7 to 14 per cent. of boulders of over one-half cubic yard, and it was necessary to take out a considerably larger amount of rock on account of seams and shakes than the test pits gave

reason to anticipate. But No. 4, which was made by a contractor of large experience is unaccountable, except on the supposition that the bidder assumed that all the rock would be in the form of boulders of from one-half to one cubic yard content and easily handled.

§ 35. "*D.* For Dimension Cut Stone Masonry," every stone to be cut to exact dimensions; the beds and joints dressed to lay ¼-inch joints, and the exposed faces hammered down to an even surface.

No. of bid	1	2	3	4	5	6	7	8
Price, per cubic yard	$30.00	$60.00	$78.84	$27.00	$40.00	$108.00	$66.66	$40.00

In this and the following masonry items, the prices offered vary with the previous experience of the contractor. No. 1 had never done similar work. Nos. 4 and 5 had never done work for the Croton Board, but were accustomed to railroad work. Nos. 2 and 3 had done some similar work for the Croton Board, and No. 6 had done a large amount of precisely similar work. No. 7's prices were high throughout. No. 8 was an engineer, who ought to have known the cost of such work better. The estimated quantity was small.

§ 36. "*E.* For Cut Range Masonry."—Certain portions of the masonry around the gate-house were specified as coming under this head; the beds to be cut to lay ¾-inch joints throughout, and the end joints for 18 inches from the face; the face to be pointed down to ½-inch projection, with a chisel draft around the arris.

No. of bid	1	2	3	4	5	6	7	8
Price, per cubic yard	$15.00	$25.00	$21.75	$20.00	$12.00	$27.00	$47.00	$22.00

§ 37. "*F.* For Rubble Range Masonry."—The face walls of the dam were specified as to be of this class. The setting-bed to be cut to lay ¾-inch joint throughout, and the ends and top for 12 inches back from the face; the rise of the stone to be not less than 12 inches, and the beds from 2½ to 4 feet; the face to be rough, with line pitched at the arris.

No. of bid	1	2	3	4	5	6	7	8
Price, per cubic yard	$11.00	$14.00	$17.00	$11.93	$9.00	$10.00	$40.00	$18.00

One important reason for the discrepancy in the prices on this item, was the title adopted in the specifications. *Rubble range* was not an ordinary term for any class of cut stone masonry, and while the specifications distinctly described the work, and the portions of the dam where it was to be used, the use of the word *rubble* appears to have produced an impression on the minds of some of the bidders, that it was to be *random* work, laid up without regular courses. It was in fact, however, laid in courses, extending the length of the dam, the rises varying from 14 to 26 inches, and averaging 21 inches.

It was afterwards stated by some of the bidders that they had intended contesting the classification of this masonry, and that they expected to be able to procure its being rated as "Cut range," and therefore had bid a low price for it. This practice of *gambling on items* is inseparable from the method of letting work at public competition to the lowest bidder. The only protection the engineer has, is to make the specification so distinct that no question can arise as to the classification of work. In the specifications for the dam, the description of this class of work is so explicit, that no claim for payment for it under any other classification, has been or can be allowed.

It might have been better if the word rubble had been omitted, as that appears to be the only basis for any misunderstanding.

§ 38. "*G.* For Concrete Masonry" laid below the level of the stream; stone to be broken to not more than 2 inches in its largest dimension, nor less than ½ inch in its smallest. Mortar to be composed of one part of hydraulic cement to two parts of sand, by measure; the proportion of mortar to stone to be such as to fill all void spaces, and be in excess of the same not more than ten per cent. The concrete to be laid in courses of 6 inches, and well rammed. In the concrete, and to be measured as part of the same, it was permitted to lay large unwrought stones. These stones to be laid in full beds of mortar, and the surface covered with mortar ½-inch thick immediately before laying concrete around them. The price for concrete to cover all expense of bailing and draining:

No. of bid	1	2	3	4	5	6	7	8
Price, per cubic yard	$8.00	$7.50	$6.60	$9.93	$10.00	$10.00	$11.45	$17.00

§ 39. "*H.* For Concrete Masonry" laid above the stream level. The specifications the same as for "*G.*" This concrete to be inclosed between the rubble range walls. The large stones inserted, to average 5 feet in length and 2 square feet in cross section, and to be laid in layers alternately parallel and at right angles to the walls of the dam.

No. of bid	1	2	3	4	5	6	7	8
Price, per cubic yard	$6.50	$7.00	$6.10	$7.93	$8.00	$10.00	$11.45	$16.00

The difference between the prices for "*G*" and for "*H*" is due to two causes. In laying "*G*," pumping and bailing would probably be needed. This would increase its cost. On the other hand, the larger area to be covered, and the freedom from the obstruction and interference caused by the laying of the cut stone facings ought to reduce the cost of laying the lower concrete, and the additional hoist, as the masonry rose above the surface of the ground, ought to increase the cost of the concrete

above stream level. Two of the bidders estimated that these variations would counterbalance each other. The others considered that the concrete below stream level was worth from $0.50 to $2.00 per cubic yard more than that above. Actually the cost of pumping chargeable to concrete was equivalent to about 18 cents per cubic yard of that below—and the hoisting to about 8 cents per cubic yard above—the stream level.

§ 40. The provision permitting the insertion of large unwrought stone caused variations in the estimates. In the absence of data as to the proportion which the stone could bear to the mass, some of the bidders assumed that from two-thirds to three-fourths of the mass would be of stone, which only required to be quarried, and could be set very quickly. In practice it was found, however, that not more than one-third of the mass could be made of the large stone. It was necessary that the stones should be far enough apart to allow the concrete to be thoroughly spread and rammed between them. This requires a space of not less than 12 inches between the nearest points, and with irregular stones the average distance is greater than this. When stones of irregular heights were laid, the leveling up necessary to prepare for a fresh layer of stone made the mean depth of concrete between the layers probably nearly 12 inches.

§ 41. The next three items, "I," "J" and "K," are for small quantities of special work, and the bids are not worthy of particular consideration, most of them having been made apparently at random, without regard to the cost of the work.

§ 42. "*L.* For Brick Masonry in Arches" of water-ways and gate-house. This item, of some 20 cubic yards, was the only one on which the bids varied less than 200 per cent.

§ 43. The contract, as awarded at this time, was for the construction of the dam to within 30 feet of the top. The appropriation authorized was not sufficient for the completion of the whole dam, and the law forbade any contract to be made in excess of the appropriation. Further expenditure was authorized subsequently, and the upper 30 feet of the dam was built under a new contract, made with the same parties that built the lower portion.

Progress of Construction.—§ 44. Work was begun in September, 1866. A new channel was made for the stream near the base of the North-east Hill, an embankment built around the remainder of the dam site, on the flat ground, and carried up 10 feet above the stream level. The pit within this embankment was partially excavated, the material

being deposited below the dam. The quarrying and cutting of stone was begun, and a Blake's stone-breaker put in operation. On October 29th–30th, 3.91 inches of rain fell in 10 hours, causing a heavy freshet which washed away the contractor's bridges, filled the pits with water, and cut embankments badly. After pumping out the pit, the excavation was continued until December 21st, when the pit was allowed to fill.

§ 45. Some observations of work done in bailing from a pool into a trough 4 feet above the water, showed that the average work per minute, of a man was 12 bucketfuls, equivalent to about 1,200 pounds of water 1 foot high.

§ 46. The winter was devoted to quarrying and breaking stone. Some cement was delivered, but was found not to be equal to the requirements of the specifications. A cargo of 900 barrels was rejected, and the delays and discussions consequent upon this prevented the receipt of any more cement until spring.

§ 47. The great distance of the work from any rail or water transportation was a serious drawback. The nearest point to which supplies could be brought by water was 16 miles distant, and over a very bad road. The nearest foundry and machine-shop was at the same distance. The nearest railroad station was 8½ miles distant, on the Harlem Railroad.

§ 48. Sand, of proper quality, was difficult to get. All the bank sand within many miles was tested and rejected as containing too much dust and loam. The only sand found at all admissible was procured from the bed of the stream, in pools where the flow was sluggish and the sand had settled. Most of this was of good quality, but some was very dirty and had to be washed, and all of it had to be screened to remove chips and coarse material. From beginning to end of the construction of the dam there was a continuous struggle on the part of the engineer to have the sand for mortar properly cleaned and screened, and of the contractor to avoid it. That sand should be sharp, clean, and screened to proper size, is absolutely essential to good mortar, and there is no point on which, as a general thing, contractors and builders are so lax. Several methods of washing were tried, of which the most successful consisted in spreading the sand in a layer of about 3 inches in depth on the bottom of a shallow box, 6 feet by 12 feet, slightly inclined, and playing upon it with a hose from a force-pump. With proper manipulation the sand was pretty thoroughly cleaned. The process was expensive, however, and after one or two loads were operated on in this way, the sand would come to the work much cleaner.

§ 49. In the spring of 1867 the main pit was pumped out, and excavation continued. The engine and boiler of 15 horse-power used for running the stone-breaker, were placed on the embankment above the pit, so as to operate a hoisting drum and an Andrew's pump throwing an 8-inch stream. After the pit had been once emptied, the pumping did not need to be continuous. The engine was able to hoist material from the pit, and at the same time either pump or break stone, but not together.

§ 50. North of the stream the foundation for the gate-house was prepared. On the side towards the stream, a trench 3 feet wide and 4 feet deep, was dug to the rock and filled with concrete, as a temporary dam to keep water from the gate-house pit. This concrete was made with 24 feet of broken stone to 1 barrel of cement and 2 of sand, and was deposited gently in the still water which filled the trench. It was not rammed, but the wet mass was worked down with shovels. One year afterwards, when this concrete was removed to make the connection with the gate-house, it was found necessary to drill and blast it. It had formed a good bond with the rock, and apparently no water had passed under it.

§ 51. The first stone of the dam was laid July 9th, 1867, in the gate-house foundation, and concreting was begun in the main pit, July 11th. During the rest of the season the foundation in the main pit was raised to the stream level, and the masonry of the gate-house and waterways was carried up 8 feet. The side of this, towards the stream, was built vertically, the concrete being put in against plank forms. On the removal of this during the following year, the face of the concrete was found good. To connect the new work with it, and avoid a straight joint through the dam, two vertical channels were cut in the concrete, 3 feet wide and a foot deep. These were cut by a stone-cutter, with a point. A pick made no impression on the concrete.

§ 52. Masonry was suspended November 5th, and the concrete already laid, covered with 2 feet of earth to protect it from frost. Some rubble masonry was laid after that date in wing walls adjoining the waterways above and below the dam. In freezing weather the mortar was mixed with salt-water. The rule for proportion of salt was one said to have been used in the works at Woolwich Arsenal some years ago, viz. : dissolve 1 pound of rock-salt in 18 gallons of water when the temperature is at 32° Fahr., and add 3 ounces of salt for every 3° lower temperature. The masonry laid with mortar thus prepared stood well, and

showed no signs of having been affected by the frost." No masonry in the dam was laid in freezing weather.

§ 53. The ensuing winter was very cold and stormy. Quarrying, cutting and breaking stone were carried forward, and sand and cement hauled. The water of the stream was carried through the gate-house.

§ 54. Work on masonry was resumed March 30th, 1868. The first part of the season was wet. A very heavy freshet occurred May 14th. The water rose 3 feet on the masonry laid in gate-house, and filled the pits. The latter part of the season was very dry, however, and the whole dam was raised 16 feet above the stream. Work was closed November 17th, and the masonry covered as before with 2 feet of earth.

§ 55. During the latter part of this year's work it had become evident that the insertion of large stone in the concrete was not beneficial. The space between the side walls was becoming narrow; at the close of work it was 21 feet. The dam was 400 feet long and 6 feet above the earth filling. It was no longer practicable to wheel the concrete from the mixing boards in barrows, but it had to be hoisted by derricks and transported by cars. When a course of facing-stone had been laid, and the space between them was to be filled with concrete, and that was filled with irregular shaped stones 1½ or 2 feet apart, it was difficult to get an even bearing for run plank and almost impracticable for a tramway. The odds were greatly against the concrete around these stones being properly distributed and rammed, if it was dumped from a car. Even when distributed by the barrow-full (2 cubic feet), it required constant attention and no small amount of persistence to secure its being spread in the narrow spaces. The stone in the vicinity of the dam which was fit for cutting had been pretty thoroughly used up, and it was questionable whether a sufficient quantity to finish the dam could be procured at all, if 3,000 additional cubic yards were put in the hearting. The scarcity of good stone caused a constantly increasing tendency on the part of the contractor to run in and cover up weather-beaten stones which were unfit for the work, on account of the impossibility of the mortar adhering properly to the weather face. It had several times occurred that such stones were brought in, and a little grout poured over them to hide the deficiencies, and very probably more of the same kind were never discovered. It was questionable, moreover, whether so great solidity and tightness could be obtained by a wall of large stones, surrounded by thin masses of concrete, as by a thick mass of concrete alone. After full discussion of the subject, it was determined to abandon for the rest

of the dam the use of the large stone, and use only concrete for the hearting. These objections to a wall of concrete and large rough stones apply mainly to thin walls, say of less than 10 feet in thickness, intended to resist water. A wall of this class may be laid to advantage when over 10 feet thick, but the stone should be required to be regular in shape, of not more than 18 inches height, and no stones should be allowed to be less than 2 feet apart. By their use under these restrictions, the specific gravity of the wall may be increased and its cost diminished, and with a much greater certainty of getting good work and tight, than if the wall is laid up with rubble masonry.

§ 56. The laying of masonry was resumed June 23d, 1869, and continued until November 1st, when the wall had reached an elevation of 38 feet above the stream. The masonry was again covered with earth 2 feet thick, which was found to protect the concrete from frost quite well. In some places it was necessary to remove from 2 to 3 inches of the old work in the spring. The winter was devoted as usual to quarrying, cutting and breaking stone. Enough broken stone was prepared to finish the dam.

§ 57. Work on masonry was resumed, April 13th, 1870, and the dam, as originally designed, was completed by November 1st, 1870. Owing to the changes of plan mentioned above, the reservoir was not ready for use until the fall of 1872. The dam contains, approximately, 21,000 cubic yards of concrete, and 6,000 cubic yards of cut stone masonry.

General Methods of Handling Materials.—§ 58. No expensive plant or machinery was used. The buildings were a contractor's office and cement house on the flat below the dam, stables and blacksmith shop near by, and a shed for stone-cutters on the flat above the dam. The stone yard was just above the dam on the north-east side of the stream. The stone for cutting and breaking for the first two years was procured from the hill-side, within 1,000 feet of the yard, and from 50 to 300 feet above it. The third year the haul was about 2,000 feet, and the last year most of the stone for cutting was hauled 2 miles. From the neighboring hills, the stones were transported on stone boats, with oxen. For long hauls, low trucks were used, the stone being first brought to the road on stone boats. When the hill-sides were dry and rocky, the wear of stone boats was enormous. For transporting the cut stone from the yard to the work, trucks drawn by horses were generally used, depositing the stone within reach of a derrick on the work. In the main pit a double-geared derrick, with 50 feet boom, was at first used

for setting stone. Its great weight made it unwieldly, and for the last two seasons derricks of 30 feet mast and 25 feet boom were used. These could readily be shifted and raised. A heavy block, with a socket for the pin at the foot of the derrick, was laid on the concrete, and boards around it for the hoisters to stand upon. The derricks were worked by hand until the masonry was 10 feet above the bank. After that 2 steam hoists were used, a vertical 8 H. P. engine, with drum, being placed on the lower side of the dam, and the fall brought to it over the edge of the masonry. All the material to be hoisted was delivered under the boom on the vertical side of the dam.

§ 59. Below the embankment level, the sand and cement for mortar were mixed dry on the bank, and sent into the pit by a shute, to the mixing board. The broken stone was sent down another shute and measured below. From the mixing board it was transported to place in barrows. When the masonry had reached a height of 6 feet above the bank, the mixing platforms were placed on the bank close to the vertical wall, the concrete shoveled from the board into boxes, which were hoisted, placed on a dumping car, running on a track of scantling resting on notched ties laid on the concrete, and wheeled to place and dumped. The track was carried along on top of the new work. As the concrete was dumped from the car, it was leveled off to about 8 inches thick and rammed. From two to three layers, according to the thickness of the course of the face walls, were put one over the other. With the track in the centre of the wall, a width of 20 feet could easily be carried forward, the car being arranged to dump in any direction. The plan worked pretty well, but was a little expensive for the contractor, as the ties became more or less imbedded in the soft concrete, and consequently there was more loose stuff to be removed in preparing for another course than if no traveling over the fresh work had occurred. Boards laid along the track, for the car-tenders to walk upon, protected the green concrete.

Stone for masonry was hoisted in the same way, and carried to the setting derricks, which were worked by hand, the reach of the boom being regulated by the topping lift, and, of course, being altered for every stone that was set. The courses were laid from one end of the dam to the other, so that, while one hoister was raising stone, the other was raising concrete for filling the space between the stones already laid at the other end of the dam.

§ 60. The stone breaker was at first placed on the bank above the

dam, and the broken stone passed from it to the meadow at the stream level. This arrangement was necessary so as to utilize the engine for pumping, but had the disadvantage of causing the stone to be hauled up again from the meadow. After pumping was no longer necessary, the breaker was placed on the hillside in such a position that it was fed from the quarry direct, and delivered stone a little above the level where it was needed, making all the haul descending.

§ 61. Wire guys were used on the derricks, and for a part of the time, when the steam hoist was used, the falls were of annealed wire rope. This generally worked well, but had one disadvantage; it would sometimes break suddenly, without warning, and it took longer to repair than rope.

§ 62. All arrangements for transportation and hoisting were under the contractor's direction, the engineer only seeing that nothing should be done in a manner detrimental to the work.

CHARACTER AND COST OF MATERIALS.—I. CEMENT.—§ 63. The specifications required that all cement should be of the best quality of American hydraulic, and subject to such tests as the engineer might direct. For this work it was not so essential that the cement should be quick setting, as that it should be strong. For concrete, a quick setting cement is not to be preferred.

The first cement, delivered in October, 1866, contained a considerable amount of coal dust, was apparently not thoroughly calcined, and, when sifted, 22½ per cent. by weight would not pass through a sieve with openings of $\frac{1}{4}$th inch. The cargo was rejected. The contractors claimed that they could not procure any better cement. All of the Rosendale works were visited, and only one company would agree to furnish a cement which should contain less than 15 per cent. of matter coarser than $\frac{1}{10}$th inch. The other companies said they had a ready sale for any cement they offered, and would not go to additional expense and trouble to grind the cement finer. The company which consented to make an effort to improve their cement furnished nearly all that was used on the dam, and, out of a great many samples tested for fineness, only three were found to equal the limit assigned. In 1869–70, samples of several different cements for sale in the New York market were tested with the results shown in the following table. Different brands are designated by different letters. "B" was the cement used on the dam.

§ 64. Tests of the tensile strength were made. The blocks had a sectional area of 2¼ square inches, with shoulders for clamps. They were

mixed in moulds, immersed in water for twenty-four hours, and broken after having stood for from seven days to a year and a half. No practical use was made of these tests at the time. The results of a large number are given in the table :—

	Fineness. Percentage by Weight.					Tensile Strength. Lbs. pr. sq. inch.			
	Brand.	Over 40.	40 to 80.	Brand.	Over 40.	40 to 80.	Brand.	7 days.	21 days.	3 mo.
1867.	A....	22.5	7.5	C....	19.4	11.2	A.....	32	94	111
	A....	16	11	C....	26.3	.....	B.....	49	111	.....
	B....	15	11	D....	15.9	11.1	B.....	61	.....	135
	B....	14	11	E....	15.4	10.9	B.....	61	84	100
1868.	B....	7.4	11.1	F....	.....	.....	B.....	58	101	.....
	B....	10	13.7	G....	22.8	.....	B.....	56	63	.....
	B....	12.5	13.7	H....	.....	.....	B.....	50	66	.....
	B....	6.6	8.4	I....	28.8	.....	C.....	37	66	75
1868.	B....	4	6	J....	7.5	.....	D.....	48	82	82
	B....	20	5.0	K....	20	.....	E.....	59	62	73
	B....	6.6	4.1	L....	18.8	.....	F.....	55	62	.....
	B....	9.4	.....	.....	.....	.....	G.....	66	94	99
	B....	15	6.9	.....	.....	.....	H.....	28	65	95
1869.	B....	10	5.3	.....	.....	.....	I.....	26	59	72
	B....	8	4.7	.....	.....	.....	J.....	54	80	87
	B....	6.2	4.7	.....	.....	.....	K.....	35	69	77
	B....	8.1	5.6	.....	.....	.....	L.....	26	74	84

§ 65. An analysis of the whole series of experiments appeared to establish these facts :—

(a.) The average tensile strength per square inch at the end of seven days, of cements, of which less than 10 per cent. by weight was over $\frac{1}{40}$th inch diameter was 54½ pounds, while that of those of which more than 15 per cent. was over $\frac{1}{40}$th inch diameter, was 39 pounds.

(b.) When cements containing from 10 to 23 per cent. of particles over $\frac{1}{40}$th inch diameter were sifted through a "40" sieve, the tensile strength of the fine portion was 20 per cent. greater than that of the cement before sifting ; but in cements containing from 23 to 30 per cent. of the coarser particles, the removal of those particles reduced the tensile strength of the cement about 25 per cent.

(c.) The tensile strength of the cements increased continuously for 9 months. Reliable observations for longer periods were not obtained. The maximum results were :—

Time.....	7 days.	14 days.	21 days.	3 mo.	6 mo.	9 mo.
Pounds per square inch.....	86.5	89	111	155	222	250

(d.) The cements which gave the best results at seven days generally maintained their superiority throughout.

§ 66. Some of the cements used which set quite slowly at first, made exceedingly hard concrete. The most compact concrete which was cut into at any time was made with a lot of cement which barely escaped condemnation on account of its setting so slowly. No samples of the brand used refused to set under water.

§ 67. A systematic testing of *all* the cement used on a work situated as this was, is very difficult. From 2,000 to 3,000 barrels were used in a month, and it sometimes happened that, owing to delays in transportation, all the cement used for several days in succession was delivered directly at the concrete boards by the teams which hauled it from the railroad. It is due to the cement company which furnished the cement to say that it was generally very satisfactory. The difficulty with most of the cement companies now is, that during the summer their works are taxed to their utmost capacity to supply the market, and, in ordinary building, almost anything which bears the Rosendale brand is accepted without question, so that there is no inducement for them to take any pains to improve the quality of their cement. The experience of Mr. Shedd on the Providence (R. I.) sewers shows, however, that it is not only practicable to make tests of all cement used on a large work, but to raise the standard of that which is furnished.

The average weight of a barrel of cement used on the dam was 284 pounds net.

II. Broken Stone for Concrete.—§ 68. Stone for concrete was broken to be not more than 2 inches in its largest dimension. A Blake's stone-breaker of 15-inch jaw was used, driven by a 15 horse-power engine. The stone which was obtained from the surface and from old fence walls in the vicinity of the work was tough and used up the jaws very fast. A moveable jaw ordinarily lasted 20 days. On one occasion, a jaw costing $70, and which had been hauled 16 miles over-night and put in the machine in the morning, was so worn by night as to be useless. With chilled jaws and careful handling, and using quarried stone instead of stone from the surface, a pair of jaws lasted much longer on the latter

part of the work. The last pair used broke about 3,000 cubic yards of stone.

As the stone passed from the breaker, it traversed an inclined screen, made of ½-inch iron rods placed parallel, about ¼-inch apart, on a slope of 1 to 1. The dust and screenings were carted away, and the stone wheeled into a pile. One, and sometimes two men were necessary on the dump, to break the larger stone as they were deposited. With a fresh set of jaws no additional breaking was needed, but as they became worn, stone in excess of the proper size would sometimes pass. The stone was delivered to the breaker by carts, having been first sledged to the proper size, about 12 inches square, by 6 inches thick. The machine, when running at full speed, with one man feeding, two men supplying him with stone, one keeping the screen clear and helping to fill barrows, two wheeling away the stone and one on the dump, could break 144 cubic feet in an hour, or at the rate of 54.4 cubic yards per day of 10 hours. This excessive speed was kept up, however, only as long as it was known that an inspector was timing it. The average rate of breaking for the last year was 3.8 cubic yards per hour, which may be assumed as the economical rate for the 15-inch machine.

The largest sized machine (20 inches) will break 8 cubic yards per hour if fed to that capacity, but 6 cubic yards per hour is more economical.

§ 69. The cost in time, of breaking stone, was as follows:

			1867.	1868.	1869.	1870.
Total cubic yards broken			2 410	4 170	5 720	3 650.
Number of days			109	152.5	155.5	96
Average cubic yards per day			22.1	27.3	36.8	38.
Days per Cubic Yard.	Furnishing.	Laborers sledging	0.269	0.322	0.224	0.410
		" loading carts		0.051	0.042	0.087
		Carts hauling	0.049	0.092	0.066	0.118
	Breaking.	Engine and machine	0.045	0.037	0.027	0.026
		Labor tending machine	0.360	0.238	0.158	0.174

The difference in sledging is accounted for thus:

1867. Many fence wall and cobble stones were used, which needed no sledging, but were hard to break.

1868. Refuse from quarry almost altogether used, which had to be sledged.

1869. Stone yard and quarry spauls used up.

1870. Stone-cutting and quarrying for same, done at a distance, and stone quarried near by for breaker; consequently nearly all stone used was sledged.

The carting varied in the same way for the same reasons.

The tending was greater in 1870 than in 1869, because in the earlier year the breaking covered the working season, during a large part of which the carts hauling stone to the concrete boards took their loads direct from the breaker, while in the winter of 1869–70, all stone broken was wheeled into the pile, the wheelers being reckoned among the *tenders*. The item for engine and machine includes cost of engine driver and helpers, fuel and repairs of engine. In estimating cost of breaking, the repairs and maintenance of stone-breaker must be added. This is worth about 0.05 day laborer per cubic yard.

The stone, when broken, weighed 90.3 pounds per cubic foot, and the void spaces averaged 47.55 per cent. of the bulk.

III. Mortar and Concrete.—§ 70. The mortar made of one barrel of cement to two of sand, measured 9.70 cubic feet per batch. This was the average of a large number of measurements, the extremes varying 7 per cent. each way. While the void spaces of the stone averaged 47½ per cent., there would sometimes be a coarse run of stone which had 53 per cent. of void spaces. To cover such cases, the mortar had to be 54 per cent. of the bulk of the stone. This makes the charge of stone to a batch of mortar 18 cubic feet. The stone was measured in barrels, and at first 4½ barrels, holding 18 cubic feet, was the charge of stone. As it was found after some use of the barrels that they became coated with dust and screenings to such an extent as to considerably diminish their capacity, 5 barrels of stone (which held the proper quantity) was the charge for a batch of concrete.

§ 71. For mixing concrete, platforms 12 by 15 feet, of 1¼-inch plank were used, the plank laid lengthwise in the direction the mixers used the shovel. Planed and matched plank are best, as causing less resistance to the shovel, and less loss of water and cement.

§ 72. The sand was wheeled or carted from the piles and deposited at one corner of the board, and 4 or 5 barrels of cement brought at a time and placed there also, the cart on its return-trip taking away the empty barrels.

The sand and cement mixer, standing a barrel with both heads out on the corner of the platform, filled it with sand, which, with a twist of the barrel he spread, then opened and emptied a barrel of cement on it, and standing the sand barrel on the pile, filled it again. He then turned the mass over and heaped it in a conical pile.

§ 73. The broken stone was deposited near the platform and washed in the pile by a hose. The stone-man shoveled it into 5 barrels with handles and with holes bored in the bottom, which stood in a row at the edge of the platform, and then poured over each from one to two buckets of water, which drained out through the holes in the bottom.

§ 74. Four "mixers" cast the sand and cement from the prepared pile on the platform so as to form a basin into which from 7 to 10 bucketfuls of water were poured from a tub standing at one end of the platform which was kept constantly filled. As the water was put in, the mixers with the backs of their shovels spread the dry stuff over it, taking care not to let the water break through the ring, and mixed the mass by running their shovels through and turning them, thus making a bed of mortar about 8 by 5 feet and 3 inches thick. On this they emptied the barrels of stone, by upsetting them over the handles, and then scraped up the loose mortar around the edges, throwing it on top.

§ 75. Then standing in pairs opposite to each other at the ends of the pile, they turned the mass outward by running the shovel under the stone and mortar, and turning it on its outer edge and drawing it back with a spreading motion over the turned mass. The opposite mixers worked in until the shovels met and then they returned to the edge again. When the two pairs met in the centre of the board, the concrete was in two piles. The operation was then reversed, and the concrete turned again towards the centre, leaving it in one pile, which was then shoveled into the hoisting box. In the meantime, the sand and stone-men had been preparing their materials for the next batch, to which the mixers proceeded as before. The whole operation was performed in from 8 to 10 minutes. In 1869, a day's work for a gang was fixed at 50 batches of concrete. In 1870, 60 batches were given as a day's work, and the task was accomplished in from 8 to 9 hours, according to the length of travel of the cars, and consequent interval between the delivery of the boxes. The work was very arduous, and none but the most powerful men could stand it. They acquired great skill in the manipulation of the concrete and worked very systematically. Part of the time a fifth mixer

was employed in turning up the edges of the pile and helping in the heavy shoveling into boxes.

During the first two years of the work, when the masonry was below or near the ground-level, the mixers shoveled the concrete into barrows which they wheeled and dumped themselves. They were paid by the day, and no limit as to amount was assigned them.

Two "rammers" took charge of the concrete as it was dumped from the barrows or cars, and spread it and then rammed it with wooden rammers.

§ 76. In the following table is given the cost *in time* of laying concrete for each season separately, and also, for comparison, the cost of laying concrete at St. Louis, Mo., as furnished by Mr. T. A. Meysenburg, when in charge of the construction of a reservoir for the St. Louis Water Works. He explains the greater cost of mixing by saying that laborers will not do nearly the same amount of work in that latitude that they will at the East.

	N. Y. Storage Reservoir.				St. Louis Reservoir.		
	1867.	1868.	1869.	1870.	1868.	1868.	1869.
Cubic yards laid	2 715	4 082	6 206	3 720	.....	.....	.....
Proportion of stone to mortar	1.65	1.85	1.85	1.85	2.5	2	.....

Days' work per cubic yard.	Mixed on Level and wheeled in.		Hoisted by Steam and run on Cars.		On Level—Wheeled in.		
	From 27 to 10 feet below surface.	From 10 below to 6 feet above surface.	From 6 to 28 feet above surface.	From 28 to 45 feet above surface.			
Mixers	0.223	0.227	0.145	0.121	0.603	0.537	0.399
Derrick and carmen	.....	.....	0.088	0.070	.....	.....	.....
Engine—hours	.....	.....	0.152	0.108	.....	.....	.....
Handling sand	.....	.....	0.065	0.071	.....	.....	.....
" stone	0.161	0.114	0.127	0.098	0.183	0.134	0.250
Carts	0.065	0.076	0.046	0.035	0.088	0.057	0.068
Ramming	0.125	0.078	0.071	0.073	0.125	0.107	0.128
Sand—cubic yards	0.428	0.416	0.420	0.420	0.373	0.411	0.444
Cement—barrels	1.446	1.420	1.425	1.428	1.259	1.394	1.500
Stone—cubic yard	0.888	0.946	0.946	0.946	0.926	0.826	1.111

To compare the four years' work, the hoisting must be left out of the account. In the first two years, the mixers did the same work that the mixers and car tenders together did in the last two years. The men handling sand and stone must be taken in connection with the carts, as the transportation from the piles was done by barrows or carts, according to the distance from the mixing boards. Calling a cart and driver equivalent to two men, we have, as day's work, per cubic yard :

	1867.	1868.	1869.	1870.
Furnishing material to mixers	0.291	0.266	0.281	0.229
Mixing and transporting to place	0.223	0.227	0.233	0.191
Total	0.514	0.493	0.514	0.420

This shows that the increased charge of stone, and the work coming nearer to the surface, diminished the cost in 1868, but that the change in 1869 to the system of paying for 50 batches as a day's work made a considerable increase in cost for that year, while in 1870, when 60 batches was a day's work, the cost was similarly diminished. One reason for the diminished cost of engine in 1870 was that, when concrete had to be put in, two gangs of mixers were kept going by one engine on short runs of the car.

§ 77. The spreading and ramming cost more the first season than afterwards, because, at the beginning of the work, the concrete was made much less moist than in the succeeding seasons, and the first layer on the rock in the bottom required the closest attention to obtain proper adhesion, and insure the bottom of the course being thoroughly filled with mortar. It was found afterwards that where the concrete was surrounded by large stones, freshly plastered, and the water was confined and prevented from running off, the mass worked better and more easily by being made quite wet. A batch transported from the mixing board to the place where it is to be laid, in one mass, and there dumped and spread to proper thickness (about 8 inches), makes far better and more homogeneous concrete than if moved in small quantities in barrows. There is less loss of mortar and water, less separation of the fine and coarse parts of the batch, and, consequently, less tendency to have part of the concrete honeycombed. When the mortar is quite dry, hard ramming is necessary to consolidate the mass, but it also tends to disintegrate

it the instant that the cement has begun to set, and this is generally before the mass can be thoroughly consolidated. The best method is to have the mortar a little more moist than is proper for masonry, the batch dumped in one mass, leveled off with shovels (the spreader at the same time turning over any irregularly mixed parts), and then slightly rammed with a wooden rammer. Where it comes against walls or forms, it is better to compact it by cutting with a shovel and then slightly ramming, than by attempting to make it solid by ramming alone. The amount of water needed varies with the temperature and condition of the atmosphere. Ordinarily, when the sand is slightly damp, and the stone well washed and wet, the quantity of water used in mixing the mortar should not exceed 3¼ cubic feet per batch.

§ 78. The concrete, when set, weighed 133¼ pounds per cubic foot; its specific gravity was 2.13.

IV. Setting Large Stone in Concrete.—§ 79. The large unwrought stones laid in the concrete, from the foundation to within 45 feet of the top of the dam, were set in full mortar beds, and the surfaces plastered just before concrete was laid around them. The setting was mostly done by laborers, one mason superintending. The cost in day's work per cubic yard was as follows:

	1867.*	1868.†
Total quantity laid, cubic yards	1 234	2 353
Percentage of whole mass	32	36.5/10
	Days, per cubic yard.	
Foreman (mason)	0.046	0.057
Laborers setting	0.208	0.142
" plastering	0.085	0.056
" mixing mortar	0.078	0.083
" at derrick	0.238	0.254
Cement—barrels	0.283	0.299
Sand—cubic yards	0.084	0.088
Stone— " "	0.897	0.892
Transporting stone to the work—teams	0.160	0.073
Laborers loading—teams	.....	0.305

* Stone lowered average of 20 feet.

† One-half lowered 5 feet; one-quarter swung in level; one-quarter hoisted 6 feet.

The cost of the mass of concrete and large stone, as laid in 1867, was 89½ per cent. of the cost of concrete alone, and in 1868 was 84½ per cent. of such cost.

§ 80. For reasons before given (§ 55) the percentage of large stone in work of this class ought not to exceed 25 per cent. of the mass, in which case the cost of the mass may be safely estimated to be reduced 10 per cent. below concrete cost, and its specific gravity increased 8 per cent.

V. Cut Stone Masonry—Quarrying.—§ 81. The stone for cutting was split out on the hill sides, and hauled to the stone yard near the dam. During the first year the quarry was not well managed, and the cost was too great to be of service for comparison. During the second year, the greater proportion of the stone quarried was for rough stone and for breaking. The cost in time of quarrying the stone not intended for cutting was for 3,400 cubic yards :

	Days per cubic yard.		Days per cubic yard.
Foreman	0.041	Tool-boy	0.035
Drillers	0.339	Teams	0.141
Laborers	0.14	Labor, loading teams	0.077
Blacksmith	0.036	Tools, powder and fuse, to be added.	

§ 82. From January, 1869, to July, 1870, 4,000 cubic yards of stone for cutting were quarried. All of this was split out with plugs and feathers, and hauled to the stone yard, an average distance of one mile. The cost in time was :

	Days per cubic yard.		Days per cubic yard.
Foreman	0.114	Tool-boy	0.108
Drillers	0.917	Teams	0.620
Laborers	0.429	Labor loading teams	0.284
Blacksmith	0.102		

VI. Stone Cutting.—§ 83. Rubble range masonry is described above in § 37. The stone was estimated at 2½ feet beds. The actual average bed as laid in walls was 2.715 feet. As estimated, 10.8 square feet of face = 1 cubic yard. The average face dimensions of the stone were 1.8 feet rise × 3.6 feet long = 6.48 square feet face per stone ; this gave about 19 superficial feet of joint cutting per stone.

The average days' work of a man, deduced from 3½ years' work, in which 5,200 cubic yards were cut, was 6.373 square feet face per day. That is, 1 cubic yard took 1.569 days' work of cutter. Average day's work of superficial feet of joint was 18.7 feet per day.

In cutting the granite for the gate-houses of the Croton Reservoir at 86th street, in New York, in 1861–2, the minimum day's work for a cutter was fixed at 15 superficial feet of joint. This included also the cutting of a chisel draft around the face of the stone, which costs per linear foot about one-fourth as much as a square foot of joint, making the actual limit equivalent to about 17.7 square feet of joint.

On the same work, the proportion to be added to the cost of cutters to give the total cost of stone yard was as follows, for 19 months work :

For superintendence	add	8 per cent.
" sheds and tools	"	7 " "
" sharpening tools	"	11 " "
" labor moving stone in yard	"	10 " "
" drillers plugging off rough faces	"	4 " "
Total to be added		40 " "

§ 84. For dimension stone, cut to ¼-inch joints, and hammer-dressed (pean hammered) on the faces, 1 cubic yard takes 9 days' work of cutters. The coping of the dam, composed of two separate courses of stone,

1 of 12-inch rise, 30-inch bed, average length 3½ feet.
1 " 24- " " 48- " " " " 2½ "

top pean hammered, face rough with chisel draft around it, beds and joints to lay ¼ inch, took per cubic yard 6.1 days' work of cutters.

Grooved work for stop plank on upper face of dam as shown in Fig. 4, with fine cut grooves, courses averaging 20 inches rise, took 10.86 days' work of cutters for every foot rise. Steps cut in rock to receive the face wall of dam on northeast end, took 1.2 days' work of cutters per linear foot of face. The above rates are all for granite, or stone of similar hardness.

The stone used for the dam averaged 172 pounds per cubic foot, and the specific gravity as laid in wall with mortar was 2.71.

VII. Laying Masonry.—§ 85. Most of the cut stone was laid by one mason. More than two were not employed at any time. The mason's gang also shifted derricks. The cost of hauling stone to work varied with position of the blocks in the yard and whether they were assorted there into courses, or lay promiscuously.

The cost in time of laying the masonry in each year was as follows :

Cut Stone Masonry.	1868.	1869.		1870.
Total cubic yards laid	1070	2270		2530
	Hand hoist.	Hand hoist.	Steam hoist	Steam hoist
Days per cubic yard.	Average 5 feet.	10 feet to 20 feet.	20 feet to 30 feet.	30 feet to 50 feet.
Mason.....	0.120	0.119	0.082	0.108
Laborers.....	0.184	0.188	0.145	0.155
Mortar mixer.....	0.100	0.082	0.076	0.101
Derrick and carmen.	0.327	0.341	0.235	0.261
Engine, hours	.....	.....	0.462	0.490
Cement, barrels.....	0.314	0.304		0.280
Sand, cubic yards....	0.093	0.093		0.093
Stone, cubic yards...	0.887	0.889		0.900
Teams from yard....	0.100	0.056		0.110
Labor loading teams.	0.184	0.223		0.086

§ 86. The stones were suspended by two hooks at the ends of a chain, on which was a sliding ring. The hooks were put in notches on the front and back of the stone, made with a stone-cutter's "point." The sliding ring enabled the bed to be slung at any angle. The lower face wall was set with the beds sloping back 2 feet in 5. This inclination caused a good deal of trouble. A stone would rest without sliding when it was first laid on the mortar, but when struck with the maul would slide unless the blow was delivered perfectly square with the stone. This was almost impossible to do, and as a good purchase for a pinch bar could not well be obtained, the masons used blocking against the concrete to hold the stone in place. This had a tendency to throw the rear of the stone up and keep it from its bed. Although apparently a small matter and easily attended to, this is in fact a serious objection to the laying of stone on that slope, when, as in this work there are over 3 miles of stone to be so laid. It is better to diminish the face batter to one-third instead of four-tenths, even if it involved the making of an occasional offset in the wall to reduce the amount of material.

VIII. Brick Masonry.—§ 87. The amount of brick masonry was small, only the arches of the waterways, and the interior walls of the gate-house being laid in brick. The bricks used were "Croton" hard-burned selected brick. The arches were semicircular, 4½ feet in diameter, laid in

three rings. The inner ring contained 33 brick, the middle one 38 and the third 42. As laid in the arch, 22 stretchers or 47 headers were 14.90 feet long. 1 cubic yard took 554 brick and 0.95 barrels of cement.

The sizes of brick vary so much in different localities that any estimate based on the number of brick laid has only a limited local value. The proportional number of brick used in different classes of wor.. may be useful. Thus, 1,000 of the same kind of brick, viz.. "Croton hard brick," laid,

In lining wall of gate-house substructure, 86th street, New York	45.2 cubic feet.
In flat arch, High Bridge enlargement, New York	46.0 "
In small arches, Storage Reservoir	48.9 "

On an average—the weight of these bricks is 4½ pounds each, and they will absorb 19 per cent. of their bulk of water.

The time of masons laying brickwork under inspection varies very much from that in ordinary building. No class of mechanics needs more instruction at the outset, and constant supervision than bricklayers on public works.

In estimating on cost of brickwork, the class of work must be particularly considered. The great difference in cost of different kinds of work is exemplified by the following table of actual cost:

	Days mason per cubic yard.
High Bridge enlargement 1863. Lining wall and flat arch laid with very close joints	0.714
Washington Aqueduct, 1864. Circular conduit 9 feet diameter, 12 inches thick	0.439
St. Louis Water Works. Conduit, semicircular arch 6 feet diameter, vertical sides and invert	0.364
Storage Reservoir, 1869. Lining of gate-house, walls and arch; rather rough work	0.304

ROADS.—§ 88. Three miles of new roads around the reservoir were built by the Department. The section of the road as built is shown in Fig. 3. The road-bed is 16 feet wide with crown of 6 inches, and a ditch 1½ feet deep on the side. All material was removed to a depth of one foot below the grade, and a top dressing of the best gravelly earth in the vicinity, free from stones over 1½ inches in diameter put on and well carted over. Fillings were made with loose rock, and frequent culverts provided. On the reservoir side, retaining walls of boulders, topped with a fence wall 3½ feet high, 2 feet wide at bottom and 18 inches at top, were built. These roads have been in use five years, have

never been repaired and are perfectly good now. The cost of the road, fence-walls and three bridges was $18,172 per mile.

The cost of the dam alone to the completion of the original design (except calking and pointing the upper face) in 1870, was $370,000. The total cost of the reservoir, including land, dam, roads and engineering to the same date was $590,000.

§ 89. In preparing this memoir, an endeavor has been made to present only the results of actual experience, and to aid in supplying a want which is often felt by engineers in estimating the cost of work with the details of which they are not familiar. Every engineer of experience has his collection of memoranda concerning work on which he has been engaged, but when called upon to estimate on another class of construction, data are very difficult to obtain. The estimates of contractors are too frequently based on mere guess-work, as is shown by the very great discrepancy in the bids on every work which is offered at public competition. The only data which can be relied upon are founded on observation of the time actually taken to perform certain operations. With these as a basis, the cost at current prices can be computed. To this cost must be added the amount needed for plant, superintendence and profits, concerning which no rule can be given, as every public work has its peculiarities which require special study, but at the bottom of all is the actual cost of doing the work, and a knowledge of this is essential to making an intelligent estimate.

CIV.

COMPOUND AND NON-COMPOUND ENGINES, STEAM-JACKETS, &c.

A Paper by CHARLES E. EMERY, M. E., Member of the Society.

PRESENTED DECEMBER 19TH, 1874.

Herewith are presented two tabular statements; No. 1, showing the results of experiments made at Baltimore, Md., in May, 1874, with the steam machinery of the U. S. Coast Survey steamer "Bache," under the general direction of the writer, and No. 2, showing the results of experiments made at the U. S. Navy Yard, Boston, Mass., in August, 1874, with the steam machinery of the U. S. revenue steamers "Rush," "Dexter" and "Dallas," under the general direction of Chief-Engineer Charles H. Loring, U. S. N., and the writer.*

The tables and occasional extracts are taken from the official reports to the Department. The results of the experiments with the Bache are published by permission of Capt. C. P. Patterson, Superintendent of the U. S. Coast Survey, in advance of the completion of the final report of investigations upon the same general subject.

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* The experiments with the "Bache" form part of an extended series of investigations with steam machinery of various kinds, made to ascertain the best means of securing economy of fuel, which were commenced by the writer (then an assistant-engineer in the U. S. Navy) in the year 1866, and continued in connection with the Novelty Iron Works, New York, where a special apparatus was fitted up for the purpose. [They were however entirely independent of those the government had in progress at the same time.] Among other results, it was in due time developed that the compound engine furnished one of the best practical means of securing economy of fuel, but the proprietors of the Novelty Iron Works decided to close the establishment in the winter of '69–'70, and nothing further was done at the time.

The results then obtained showed, with considerable accuracy, the law of variation in the cost of the power due to changes of the steam pressure and degree of expansion, and other matters of importance, which, coming to the attention of Capt. C. P. Patterson, then connected with, now superintendent of, the U. S. Coast Survey, so interested him that he made arrangements to provide the means required to complete the trials on the plan originally intended. The experimental machinery was, in part, purchased in the general sale at the Novelty Iron Works, and after being reconstructed the experiments were in due time proceeded with, though the loss of the skilled workmen and facilities of the Novelty Works was severely felt and caused unexpected expense and delay. Messrs. Hecker & Bro., the well-known millers, with characteristic public spirit, rendered valuable assistance by providing a location for the machinery, with use of boilers, pumps, &c.

These experiments included a trial of nearly every possible change of arrangement and condition to which simple and compound engines could be put in relation to steam pressure, expansion, use of steam-jackets, &c. Upon completing the principal computations, it was found desirable to ascertain the nature of the change in result due to increasing the size of engines of the same general character. Other experiments with which we had been associated furnished part of the necessary data, but to still further complete the investigation, with the consent of the Superintendent of the Coast Survey, practical experiments were made with the machinery of the steamers Blake and Bache, corresponding to some of those with the experimental apparatus.

The pressure of professional duties has prevented the writer from promptly completing the arrangement and discussion of the many hundred experiments, with the deductions due to the numerous changes of condition, but the results will, as soon as possible, be reported to the Superintendent of the Coast Survey and promptly published.

The information derived from the trials of the experimental machinery was utilized in some degree in designing the machinery for the U. S. Revenue steamer Rush, and it having

DESCRIPTION OF THE MACHINERY.—The engine and hull of the steamer Bache were built in 1870, by Messrs. Pusey, Jones & Co., of Wilmington, Delaware. The engine was designed by the writer, and is of the steepled compound type, the larger cylinder, which is steam-jacketed, being supported vertically upon frames as in ordinary vertical engines. The smaller cylinder, which is not steam-jacketed, is supported above the other by 4 side columns. The pistons are attached to the same piston-rod. Suitable pipes and valves are provided so that the live steam can be supplied to the larger cylinder and excluded from the smaller, the former then working as a single engine. Ordinarily, when operating as a compound engine, the steam from the upper cylinder passes to the chest of the lower through a large pipe, no pains having been taken to reduce the intermediate space, as the distribution of power between the two cylinders can easily be regulated by the adjustable cut-off on the larger.

Steam is distributed to each cylinder by a short slide valve at each end, and for both cylinders there are independently adjustable cut-off plates on the backs of the main valves. The valve faces of upper cylinder are carried out so that the valves of both cylinders are operated by continuous stems, but the ports, which lead directly out from the clearances, are shorter than usual for the upper cylinder, and of the least possible length in the lower cylinder. The engine is provided with a surface condenser. The air pump is operated through the usual levers from the main crosshead. The circulating pump is of the centrifugal pattern, operated by a small independent engine, directly connected.

The boiler is of the Scotch return tubular type, and of sufficient strength for a steam pressure of 100 pounds, according to U. S. laws, though ordinarily worked at 80 pounds pressure. It is provided with a steam chimney, arranged above the front connection in the usual manner, and connected to boiler by a large tube.

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been found expedient to put engines of different kinds in the two sister vessels, the Dexter and Dallas, the opportunity presented of testing, in these vessels, the relative merits of the three kinds of engines attracted considerable attention. Several manufacturers and engineers expressed a desire that competitive trials be made. A correspondence on the subject was opened between the Navy and Treasury Departments, which resulted in an agreement for a trial under the direction of persons representing both services, and Chief-Engineer Chas. H. Loring, U. S. N., and the writer, were selected in behalf of the Navy and Treasury Departments, respectively, to make preparations for and take general charge of the trials.

When the preparations were complete, the following officers were detailed to conduct the experiments, viz.: Chief-Engineers Edward Farmer and George D. Emmons, U. S. N.; Chief-Engineers F. H. Pulsifer and F. A. D. Bremon, U. S. R. M. As assistants to these gentlemen there were detailed Past Assistant-Engineers Harvey and Cook, and Assistant-Engineer Tobin, U. S. N., also Mr. E. Hugentobler, who was one of the assistants of the writer during the previous experiments with the steamers Blake and Bache.

During the experiments with the revenue steamers the care of the machinery was intrusted to the engineers of the respective vessels. The chief-engineers detailed for the experiments stood regular watches, with an assistant, while the experiments were in progress, and at the close certified duplicate copies of the logs, which are deposited in the Navy and Treasury Departments, respectively.

The following are the principal dimensions of the machinery of the Bache.

ENGINE.

	High Pressure.	Low Pressure.
Diameter of Cylinders, inches	15.98	25.
Diameter of Piston Rods, "	2.5	3.625
Stroke of Pistons, "	24.00	24.000
Size of Cylinder Ports, "	9 x 1.5	18 x 1.5
Ratio of Piston Displacement to Capacity of Clearances and Passages	.0486	.0405
Comparative effective Capacities of the two Cylinders	{ 1. .40987	2.4398 1.
Ratio Capacities of Cylinders to Capacity of Intermediate Chamber and Passages	2.6272	1.0768

BOILER.

Diameter	8 ft. 2 inches.
Length	12 ft.
Inside Diameter of Furnace Flues	34 inches.
Tubes, 90 in number, 9 ft. 9 ins. long, and 3 ins. in diameter.	
Grate Surface	31.16 sq. ft.
Cross Area of Tubes for Draft	3.78 "
Water heating Surface	950.10 "
Steam heating Surface	54.32 "
Ratio cross Area of Tubes to Grate Surface	8.25 "
Ratio Grate to heating Surface	30.50 "
Ratio cross Area of Tubes to heating Surface	252.02 "
Ratio cross Area of Tubes to Steam Space (9 ins. water above tubes)	48.80 "

The three revenue steamers are similar as respects the hulls, the screws and the boilers, but the engines are different each from the other —that of the Rush being a compound engine; that of the Dexter a high-pressure condensing engine, and that of the Dallas a low-pressure condensing engine.

The vessels are each 140 feet long over all. 129½ feet between perpendiculars at water line, 23 feet extreme breadth of beam, and 10 feet depth of hold. The draught of water aft is about 8 feet 10 inches. The hulls are of wood. The vessels represent the smallest type of full-powered screw revenue cutters adapted for cruising purposes. They were all intended to be rigged as schooners, but it having been decided to send the Rush to the Pacific coast she was rigged as a top-sail schooner. One of the vessels averaged upward of 11 nautical miles per hour, for 6 consecutive hours, on her trial trip, and neither of them averaged less than 10 knots, the machinery being entirely new in each case.

Each vessel has one boiler, 11 feet wide on base, and 9 feet high, with a double segmental shell, each portion being 6 feet 2 inches in diameter. There are 3 furnaces in each boiler, located between water legs attached to the bottom of the shell. The products of combustion return

through tubes within the shell. The boiler of the Dallas, designed for low-pressure steam, is 13 feet 9 inches long, the front connection being built in and the steam chimney attached to the boiler. The boilers of the two other vessels were designed for high-pressure steam, and are each 12 feet long, independent of front connection, which is a separate structure, bolted on. The steam chimney is also a separate structure, connected to boiler by a large tube. The boiler of the Dallas has 160 tubes, 3¼ inches in diameter, and 9 feet 3 inches long. The boilers of the two other vessels have each 158 tubes, 3¼ inches in diameter, and 9 feet 8 inches long.

The Rush is propelled by a compound engine, with vertical cylinders and intermediate receiver, arranged fore and aft at the same level, the pistons being separately connected to cranks at right angles. The cylinders are thoroughly steam-jacketed, felted and lagged, and are, respectively, 24 and 38 inches in diameter, with 27 inches stroke of pistons. The steam is distributed to the high-pressure cylinder by a short slide-valve with adjustable cut-off plates sliding on back of same. The distribution of steam to the low-pressure cylinder is effected by means of a double-ported slide-valve with lap proportioned to cut off the steam at about half stroke. The surface condenser is arranged on the starboard side. It supports 2 main columns from the cylinders, and contains 900 square feet of condensing surface. The air pump is operated from the cross-head of the low-pressure engine. The circulating pump is of the centrifugal type, operated by a small engine, directly connected. The screw is 8 feet 9 inches in diameter, with mean pitch of 14½ feet. The engine was intended to be operated regularly with a steam pressure of 80 pounds, but during the trials, hereafter referred to, it was reduced to correspond to the pressure carried on the trial of the Dexter. The machinery was designed by the writer, and built by the Atlantic Works (East Boston, Mass.), the contractors for the vessel complete.

The Dexter was also built under contract with the Atlantic Works. The engine of this vessel was built from designs of that establishment, and is of the inverted type, with a single cylinder, 26 inches in diameter, and 36 inches stroke of piston. The cylinder is not jacketed, but is carefully felted and lagged. Steam is distributed by a short slide-valve, with adjustable cut-off plates sliding on back of same. The condenser is located outside the frame, but it and the air and circulating pumps are exact duplicates of those in the Rush. The engine and boiler are designed to be operated with a maximum steam pressure of 70 pounds.

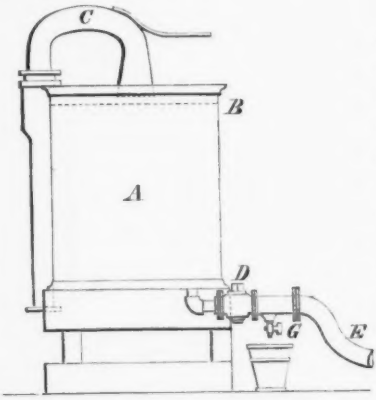
The Dallas was built under contract with the Portland Machine Works (Portland, Maine). The engine was designed in that establishment, and is of the inverted type, with a single cylinder, 36 inches in diameter, with 30 inches stroke of piston. The cylinder is not steam-jacketed, but is carefully covered with non-conducting composition and lagged. Steam is distributed by a short slide-valve, with adjustable cut-off plates sliding on back of same. The surface condenser is located under starboard frames, and has the same condensing surface as those in the other vessels. The air and circulating pumps are also substantially the same. The maximum steam pressure to be used is 40 pounds.

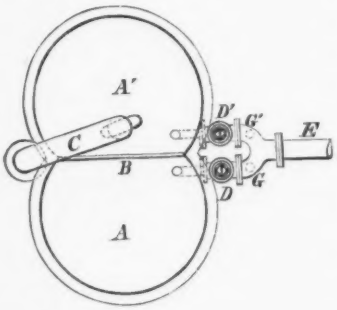
The steam jacket on the larger cylinder of the steamer Bache, and that on each of the cylinders of the steamer Rush, were supplied with steam in the following manner: steam was first admitted to the cavity in cylinder cover, from which, by means of a pipe leading from the bottom of the cavity, it was conducted to the side jacket, thereby keeping the cavity in cover clear of water. The side and bottom jackets communicated, and the water of condensation was blown from latter into the hot well, the flow being regulated, by inspection of a water level, in a glass gauge, on a small chamber in the drain-pipe.

On the Bache, when operated as a compound engine, the steam for the jacket was taken from bottom of steam-chest of upper cylinder, thereby keeping that drained. When the lower cylinder was operated as a single engine, the steam for jacket was taken from main steam-pipe. On the Rush, the steam for jackets was taken directly from the boiler.

Manner of Conducting the Experiments.—Both series of trials were made with the vessels secured to the dock. During the experiments with the steamer Bache, a double tank of iron (see next page) for measuring the water delivered from surface condenser, was placed on the main deck, in the gangway abreast of engine, and the feed water delivered to same directly by the air pump, through a pipe with flexible termination (C), which could be directed over either compartment (A, A^1) of tank. A pipe (E) with two branches (G, G^1) and regulating cock (D, D^1) in each, conducted the water from the two compartments respectively, to a tank in the hold, from which the water was withdrawn by the engine feed pumps. Each compartment held, by calculation, very nearly 10 cubic feet, at the height of a central overflow partition (B), but the exact capacity was ascertained by weighing water into same of the average temperature of the feed.

The measurement was made by filling one compartment, (A) for instance, till it over flowed into the other (A^1), which had been previously emptied; the supply was then changed to the latter (A^1), and when the surplus water in first (A) had run off, that compartment was emptied and cock on bottom of same closed in time to receive the overflow from the other (A^1); the operation being repeated alternately with each compartment.





To prevent the misplacing of the cocks (D and D^1) the attendant gave notice to the officer of the watch when each compartment was nearly full, and when the water first broke over the partition (B), a signal was given and the reading of engine-counter noted. A comparison of the differences of successive readings and the constant attendance of two persons effectually prevented errors. On the next even minute after filling a tank, the reading of engine-counter was again taken; also the usual engine-room data, and the duration of experiments was fixed, so far as measurements were concerned, by the tank intervals, and in respect to speed, by the nearest time intervals.

By this plan the officer on watch had but one thing to do at a time; the principal calculations were left for office work; the experiment could be held to start and stop at such records as best showed the uniformity of condition desired for an experiment, and an interrupted run be accurately calculated up to the end of the tank interval immediately preceding the interruption.

The same system, with some slight modification of details, was adopted for measuring the feed water during the experiments with the revenue steamers. The measuring tank was placed on the hurricane deck, for each of these vessels, and one of the engine feed-pumps employed to raise the water from hot well to a small tank placed over the measuring tank. The water from small tank was distributed in turn to each compartment of the measuring tank by means of suitable valves.

A valve on the bottom of each compartment of measuring tank was connected by pipes to the suction of one of the engine feed-pumps, by which means the water was returned to the boiler as soon as it was measured. Each valve in turn was adjusted to keep the feed substantially uniform, care being taken to empty one compartment a little sooner than the other was filled.

Each compartment of tank was found by trial to contain 1,129½ pounds of water, at the temperature of 72° Fahr. In making the calculations of the weight of the feed-water, the observed temperatures for the several experiments were taken into consideration.

On each of the vessels one experiment was made of sufficient length to determine accurately the evaporative efficiency of the boiler; and it having been shown that the water measurement was substantially the same from hour to hour, when the conditions remained uniform, the opportunity was embraced to try a number of experiments of short duration, showing the results under varied conditions, using the water measurement only. On the steamer Bache previous to the evaporation trial, a quantity of anthracite coal of fair quality, and of egg size, was, in the presence of an officer, measured into a distinct portion of the coal-bunkers and the remainder of the bunkers boarded up. From this measured quantity the coal was taken as required for the fires, and each bucket made to balance accurately a fixed weight on a scale. The weighing was done by a careful machinist, who had no knowledge of the quantity originally measured.

In the first experiment tried in this way, the two methods of determining the quantity did not agree, and the coal measurement was rejected. Upon repeating the experiment there was a very close agreement, so the weighed quantity was adopted as correct.

During the experiments with the revenue steamers the coal, which was anthracite of fair quality, was broken on the wharf to proper size (the vessels' bunkers having been closed and sealed) and filled into bags to a certain weight.

The bags were sent on board when ordered by the senior engineer on watch, he making record on the log of the number of bags and the time of receipt; a similar record being made by one of the men on the wharf. At the end of the hour the number of bags of coal actually put on the fire was reported from the fire-room and entered in the appropriate column. The several records agreed with each other, and the total amount expended corresponded with the total number of bags filled on the wharf. The ashes were measured into buckets of which the mean weight was ascertained and tallied as they were hoisted out. They were afterward weighed in gross on the wharf, and the two accounts found to agree substantially.

On all the vessels the water level in the boiler was noted every time a tank was filled. On the Bache, the water level did not vary appreciably, as the condenser was quite tight and all leaks were of a trifling nature. The same was true on the Dallas. During the trials of the Rush and Dexter, the water lost from ordinary causes in the circulation to and from the engine and boiler was replaced by running hydrant water into the tank that was being filled. The additional water was therefore measured and charged in the cost.

The loss of water was not sufficient to affect the result materially in either case. It was greatest in the Dexter, which had been on service. The safety valve of this vessel leaked slightly, and there was probably some other trifling leak that could not be detected. The number of inches that the water fell in the boiler between the periods of supply being shown in the logs, were added together, and from the same and known dimensions of boiler the volume and weight lost were ascertained quite accurately. The reduction in the number of revolutions per tank, when the water was being received from the hydrant, furnished another, and, perhaps, still more accurate means of ascertaining the proportionate amount lost and returned. The two methods closely agreed in fixing the loss in the case of the Dexter at 4.96 per cent. of the total amount of water used.

On the Bache the condensed water from the jacket and intermediate chamber was collected and weighed in separate vessels, and emptied into the compartment of measuring tank from which the boiler was at the time being fed.

A number of indicators were tested with steam before the trials, and a pair selected for use which proved correct by a standard gauge at varying pressures. Indicator diagrams were taken every twenty minutes through-

out both series of trials, and the data for the usual columns of the log, except the coal and ashes, every half hour.

It was found that the pistons of the Dexter and Dallas were tight, by removing the cylinder covers and letting on full steam pressure.

EXPLANATION OF THE TABLES.—The results of the two series of experiments are shown in Tables Nos. 1 and 2, hereto annexed. Referring to Table No. 1, showing the results of the experiments with the steamer Bache it will be observed that the experiments are arranged by the degree of expansion, under four general titles, two referring to the engine when compounded, and two to experiments with larger cylinder, when used separately—trials being made in each case at different degrees of expansion, with and without using the steam jackets on the larger cylinder. All the experiments, except the last, were made with an approximate steam pressure of 80 pounds.

The actual performances will be found in lines 46 to 58, inclusive, the previous lines showing the several observed and calculated quantities upon which the performances are based. The weights of water withdrawn from the jackets and intermediate chamber are separately set forth, also the percentage which these quantities form of the total water used. The water collected from jackets and intermediate chamber having been evaporated in the boiler, is, in all cases, included in making up line 35, showing the total weight of water used per hour, upon which the cost of the power is based.

Referring to Table No. 2, showing the results of the experiments with the revenue steamers, it will be seen that the general arrangement is substantially the same as in Table No. 1.

The actual performances will be found in lines 53 to 68, inclusive, of the table, the previous lines showing the several observed and computed quantities from which the performances were calculated.

On these vessels the evaporation experiments were made at the maximum power obtainable with the vessel at the dock. The report states: "that the evaporation was fully equal to that obtained in ordinary practice, but inasmuch as on land, and occasionally in steamers where space will permit, it is the practice to use a slower rate of combustion in comparatively larger boilers, thereby increasing the evaporative effect, there has been added to the table, for comparison under such circumstances, lines 69 and 70, showing the performances compared on the basis of the water actually used, but with boilers of such proportions or using such variety of coal that the evaporation will equal nine and ten pounds, respectively, per pound of coal."

TABLE NO. 1, SHOWING THE RESULTS OF EXPERIMENTS WITH THE STEAM MACHINERY OF THE U

		MANNER IN WHICH ENGINE WAS OPERATED.	COMPOUND, WITHOUT USING STEAM JACKET, BUT DRAINING INTERMEDIATE CHAMBER.		
1	Designation of Runs	Number for reference	1	2	3
2		Designation in log	8	7	6
3	Time	Date of experiment. 1874	May 14.	May 14.	May 14.
4		Duration of experiment hours	1.833	2.066	2.133
			H. P. Cyl. | L. P. Cyl.	H. P. Cyl. | L. P. Cyl.	H. P. Cyl. | L. P. Cyl
5	Steam Cut-off and Expansion	Average steam pressure in boiler lbs.	82.00	80.28	80.3125
6		" cut-off in fractions of stroke "	.231 | .503	.335 | .833	.405 | .779
7		Ratio of expansion	3.75 | 9.146	2.73 | 6.658	2.31 | 5.634
8	Vacuum and Barometer	Average vacuum in condenser inches	24.00	24.321	24.656
9		" barometer "	30.00	30.00	30.00
10		" " lbs.	14.72	14.72	14.72
11	Temperatures (Fahrenheit Scale)	Air. Average temperature engine room degrees	96.00	96.00	96.00
12		" " fire room "	84.00	86.33	86.33
13		Water. " " sea water "	62.00	62.00	62.00
14		" " discharge water "	87.00	92.00	91.33
15		" " hot well "	98.00	100.00	105.33
16	Revolutions	Total revolutions	4685.	5912.	6305.
17		Average " per hour	2555.91	2861.56	2955.93
18		" " per minute	42.5985	47.6927	49.2655
19	Indicator Diagrams	Average initial pressure in cylinder above atmosphere lbs.	75.00 | 1.00	76.33 | 2.458	73.00 | 3.9
20		" total initial pressure in cylinder "	89.72 | 15.72	91.05 | 17.178	87.72 | 18.62
21		" " terminal " "	24.345 | 7.72	30.47 | 9.18	32.32 | 10.02
22		" " cushion " "	28.595 | 5.532	29.52 | 5.97	32.42 | 6.02
23		" back pressure above atmosphere at beginning of stroke, small cylinder "	2.800 |	4.333 |	5.280 |
24		" total back pressure at half stroke, large cylinder "	| 3.360	| 3.414	| 3.328
25		" mean effective pressure "	34.801 | 8.153	43.514 | 9.7388	45.137 | 11.275
26		Estimated friction pressure "	.75 | 2.25	.75 | 2.25	.75 | 2.25
27	Power	Indicated horse-power (effective)	35.586 | Both. 55.928	49.85 | Both. 77.06	5332. | Both. 85.81
28		Net horse-power	34.82 | Both. 49.548	48.874 | Both. 69.812	52.434 | Both. 78.447
29	Water	Total weight of water received from hot well lbs.	2391.00	3586.50	4184.25
30		" " " " " small chest and steam jacket "			
31		" " " " " intermediate chamber "	36.00	79.00	62.00
32		" " " ascertained by measurement "	2427.00	3665.50	4246.25
33		Revolutions using same	4678.	5909.	6301.
34		Time using same hours	1.826	2.0649	2.132
35		Hourly quant's. Water per hour as ascertained by measurement lbs.	1329.13	1775.14	1991.67
36		" " " " " indicator diagrams "	862.046 | 710.228	1207.6 | 948.82	1318.97 | 1059.45
37		Proportional quantities. Per centage of total water received from jacket and small chest			
38		" " " " " " intermediate chamber	.015	.0215	.014
39		" " " " " " jacket, chest & "	.015	.0215	.014
40		" " " " accounted for by indicator	.64857 | .53435	.68028 | .5345	.66221 | .531
41	Coal and Refuse	Total coal consumed lbs.			
42		Average coal consumed per hour "			
43		Coal consumed per square foot of grate per hour "			
44		Percentage refuse from coal			
45		Combustible per hour lbs.			
46	Performance of Engine	Water per indicated horse-power per hour by measurement lbs.	23.765	23.036	23.21
47		" " " " " " indicator "	15.413 | 12.699	15.671 | 12.312	15.371 | 12.346
48		" " net " " " measurement "	26.8251	25.4274	25.3867
49	Performance of Engine and Boiler	Coal per indicated horse-power per hour lbs.	Calculated. 2.002	Calculated. 2.522	Calculated. 2.541
50		" " net " " " "			
51		Combustible per indicated horse-power per hour "			
52		" " net " " " "			
53	Performance of Boiler	No. lbs. water evap'd per lb. coal at observed pressure and temp. lbs.			
54		Equivalent evaporation from atmospheric pressure and temp. 100° "			
55		" " " " " " 212° "			
56		No. lbs. water evap'd per lb. combustible at obs'v'd press. and temp. "			
57		Equivalent evaporation from atmospheric pressure and temp. 100° "			
58		" " " " " " 212° "			

BUT

COMPOUND, USING STEAM JACKET, AND DRAINING INTERMEDIATE CHAMBER.

	4	5	6	7	8	9	10
	5	4	3	2	1st Evap.	2d Evap.	1
	May 12, 1.733	May 12. 2.066	May 12. 1.933	May 12. 1.983	May 14. 7.066	May 15, 15.233	May 12. 2.00
	H. P. Cyl. | L. P. Cyl. 81.375	H. P. Cyl. | L. P. Cyl. 80.333	H. P. Cyl. | L. P. Cyl. 80. 214	H. P. Cyl. | L. P. Cyl. 80.125	H. P. Cyl. | L. P. Cyl. 79.96	H. P. Cyl. | L. P. Cyl. 79.7	H. P. Cyl. | L. P. Cyl. 79.1
	.103 | .820	.229 | 8 31	.318 | .831	.396 | .803	.398 | .822	.4519 | .735	.554 | .795
	6.91 | 16.85	3.77 | 9.19	2.86 | 6.975	2.35 | 5.732	2.34 | 5.707	2.09 | 5.097	1.74 | 4.24
	24.50	26.50	26.50	26.562	26.115	24.39	26.5
	30.34	30.34	30.34	30.34	30.00	29.15	30.34
	14.89	14.89	14.89	14.89	14.72	14.31	14.89
	81.375	80.00	80.00	76.66	83.70	90.00	78.00
			78.00	75.33	83.80	85.06	76.66
	60.00	60.00	60.00	60.00	61.90	64.66	60.00
	82.00	79.00	78.50	79.66	90.20	92.53	80.66
	110.00	104.00	103.00	102.00	106.60	125.26	115.33
	4042.	5973.	6168.	6705.	23568.	49008.	7288.
	2332.37	2891.09	3190.89	3381.	3335.4	3217.22	3634.
	38.873	48.185	53.1815	56.34	55.59	53.62	60.56
3.9	75.75 | 1.00	77.125 | 4.00	78.58 | 6.25	75.41 | 8.66	75.32 | 8.96	74.47 | 7.52	72.41 | 13.62
18.62	90.64 | 15.89	92.015 | 18.89	93.47 | 21.14	90.30 | 23.55	90.04 | 23.68	88.78 | 22.83	87.30 | 28.51
10.02	20.76 | 10.39	27.39 | 11.89	33.72 | 12.93	38.30 | 14.23	36.95 | 14.31	34.40 | 12.93	47.05 | 16.89
6.02	26.39 | 6.77	30.03 | 4.89	36.29 | 4.89	35.05 | 4.29	40.94 | 5.02	37.69 | 6.26	42.89 | 4.89
	1.700 |	5.000 |	7.700 |	9.266 |	10.066 |	9.200 |	14.733 |
3.328	3.690	3.050	3.264	3.328	3.360	3.253	3.317
11.2756	24.484 | 10.341	35.39 | 13.014	42.34 | 14.484	42.938 | 15.88	45.3748 | 13.961	43.0608 | 14.908	45.33 | 19.34
2.25	.75 | 2.25	.75 | 2.25	.75 | 2.25	.75 | 2.25	.75 | 2.25	.75 | 2.25	.75 | 2.25
Both.	Both.	Both.	Both.	Both.	Both.	Both.	Both.
85.81	22.852 | 46.4	40.87 | 77.453	54.06 | 99.181	58.086 | 110.514	60.562 | 106.028	55.4372 | 102.263	65.911 | 134.53
Both.	Both.	Both.	Both.	Both.	Both.	Both.	Both.
78.447	22.162 | 40.59	40.005 | 70.335	53.104 | 91.216	57.068 | 102.069	59.5617 | 97.7014	54.4716 | 94.23	65.112 | 125.75
	1793.25	2988.75	3586.50	4184.25	15302.40	32278.50	5379.75
	132.25	235.25	199.00	209.	889.25	1977.00	232.00
	95.75	91.00	98.00	70.50	249.00	569.25	77.00
	2021.25	3315.00	3883.50	4463.75	16440.65	34854.75	5688.75
	4047.	5974.	6145.	6706.	23547.	49001.	7259.
	1.735	2.06631	1.9258	1.954	7.059	15.230	1.9975
	1164.9	1604.31	2016.56	2284.42	2329.03	2288.558	2847.9
1059.45	673.673 | 857.824	1092.47 | 1220.36	1485.23 | 1464.9	1778.07 | 1710.17	1681.00 | 1696.95	1518.76 | 1485.66	2323.1 | 2180.61
	.065	.070	.051	.046	.054	.056	.040
	.047	.030	.025	.015	.015	.016	.010
	.112	.100	.076	.061	.069	.072	.050
.53194	.57964 | .73809	.68095 | .76067	.73651 | .72643	.77835 | .74888	.72176 | .72850	.66363 | .64916	.81572 | .76569
						3818.	
						250.64	
						8.84	
						19.8	
						201.01	
12.346	25.1082	20.7133	20.332	20.3648	21.9661	22.3798	21.169
87	14.553 | 18.532	14.104 | 15.756	14.974 | 14.769	15.851 | 15.251	15.854 | 16.004	14.852 | 14.528	17.268 | 16.209
	28.698	22.8095	22.1074	21.9989	23.8385	24.287	22.647
ated.	Calculated. 2.749	Calculated. 2.268	Calculated. 2.227	Calculated. 2.230	Calculated. 2.405	Actual. 2.451	Calculated. 2.318
						2.660	
						1.966	
						2.133	
						9.131	
						9.206	
						10.281	
						11.385	
						11.479	
						12.830	

COMPOUND, USING STEAM JACKET, AND DRAINING INTERMEDIATE CHAMBER

T	4	5	6	7	8	9
	4	5	6	7	8	9
	5	4	3	2	1st Evap.	2d Eva
	May 12, 1.733	May 12. 2.066	May 12. 1.933	May 12. 1.983	May 14. 7.066	May 1 15.32
Cyl.	H. P. Cyl. | L. P. Cyl.	H. P. Cyl. | L. P. Cyl.	H. P. Cyl. | L. P. Cyl.	H. P. Cyl. | L. P. Cyl.	H. P. Cyl. | L. P. Cyl.	H. P. Cyl. | L
	81.375	80.333	80. 214	80.125	79.96	79.7
779	.103 | .829	.229 | 8 31	.318 | .831	.396 | .803	.398 | .822	.4519
634	6.91 | 16.85	3.77 | 9.19	2.86 | 6.975	2.35 | 5.732	2.34 | 5.707	2.69
	24.50	26.50	26.50	26.502	26.115	24.39
	30.34	30.34	30.34	30.34	30.60	29.15
	14.89	14.89	14.89	14.89	14.72	14.31
	.....	.....	80.00	76.66	83.70	90.08
	81.375	80.00	78.00	75.33	83.80	85.00
	60.00	60.00	60.00	60.00	61.90	64.66
	82.00	79.00	78.50	79.66	90.20	92.53
	110.00	104.00	103.00	102.00	106.60	123.26
	4042.	5973.	6168.	6705.	23568.	49008.
	2332.37	2891.09	3190.89	3381.	3335.4	3217.28
	38.873	48.185	53.1815	56.34	55.59	53.63
.9	75.75 | 1.00	77.125 | 4.00	78.58 | 6.25	75.41 | 8.66	75.32 | 8.90	74.47
.62	90.64 | 15.89	92.015 | 18.89	93.47 | 21.14	90.30 | 23.55	90.04 | 23.68	88.78
.02	20.76 | 10.39	27.39 | 11.89	33.72 | 12.93	38.30 | 14.23	36.95 | 14.31	34.40
.02	26.39 | 6.77	30.03 | 4.89	36.29 | 4.89	35.05 | 4.29	40.94 | 5.02	37.69
.....	1.700 |	5.000 |	7.700 |	9.266 |	10.066 |	9.200
328	..... | 3.690	..... | 3.050	..... | 3.264	..... | 3.328	..... | 3.360	.....
2756	24.484 | 10.341	35.39 | 13.014	42.34 | 14.484	42.938 | 15.88	45.3748 | 13.961	43.0608
25	.75 | 2.25	.75 | 2.25	.75 | 2.25	.75 | 2.25	.75 | 2.25	.75
th.	Both.	Both.	Both.	Both.	Both.	
.81	22.852 | 46.4	40.87 | 77.453	54.06 | 99.181	58.086 | 110.514	60.562 | 106.028	55.4372
th.	Both.	Both.	Both.	Both.	Both.	
.447	22.162 | 40.59	40.005 | 70.335	53.104 | 91.216	57.068 | 102.069	59.5617 | 97.7014	54.4716
.....	1793.25	2888.75	3586.50	4184.25	15302.40	32278.5
	132.25	235.25	199.00	200.	889.25	1977.0
	95.75	91.00	88.00	70.50	249.00	569.2
	2021.25	3315.00	3883.50	4463.75	16440.65	34854.7
	4047.	5974.	6145.	6706.	23547.	49001.
	1.735	2.06631	1.9258	1.954	7.059	15.2
	1164.9	1604.31	2016.56	2284.42	2329.03	2288.5
.45	673.673 | 857.824	1092.47 | 1220.36	1485.23 | 1464.9	1778.07 | 1710.17	1681.00 | 1696.95	1518.76 |
.....	.065	.070	.051	.046	.054	.0
	.047	.030	.025	.015	.015	.0
	.112	.100	.076	.061	.069	.0
.53194	.57964 | .73809	.68095 | .76067	.73651 | .72643	.77835 | .74888	.72176 | .72850	.66963 |
.....	.....	.....	.....	.....	.....	3818.
.....	.....	.....	.....	.....	.....	250.6
.....	.....	.....	.....	.....	.....	8.8
.....	.....	.....	.....	.....	.....	19.8
.....	.....	.....	.....	.....	.....	201.0
	25.1082	20.7133	20.332	20.3648	21.9661	22.3
346	14.553 | 18.532	14.104 | 15.756	14.974 | 14.769	15.851 | 15.251	15.854 | 16.004	14.852
	28.698	22.8095	22.1074	21.9989	23.8385	24.2
	Calculated. 2.749	Calculated. 2.268	Calculated. 2.227	Calculated. 2.330	Calculated. 2.405	Actu 2.4
.....	.....	.....	.....	.....	.....	2.6
.....	.....	.....	.....	.....	.....	1.9
.....	.....	.....	.....	.....	.....	2.1
.....	.....	.....	.....	.....	.....	9.1
.....	.....	.....	.....	.....	.....	9.5
.....	.....	.....	.....	.....	.....	10.3
.....	.....	.....	.....	.....	.....	11.3
.....	.....	.....	.....	.....	.....	11.4
.....	.....	.....	.....	.....	.....	12.6

(FOR DIMENSIONS OF ENGINE AND BOILER, SEE SEPARATE TABLE.)

AMBER.				SINGLE ENGINE, WITHOUT USING STEAM JACKET.			SINGLE ENGINE, USING STEAM JACKET.			
9		10		11	12	13	14	15	16	17
2d Evap.		1		14	13	12	11	10	9	15
May 15.		May 12.		May 18.	May 18.	May 18.	May 18.	May 18.	May 18.	May 18.
15.233		2.00		1.80	1.983	2.05	2.10	1.688	2.116	1.883
P. Cyl.	L. P. Cyl.	H. P. Cyl.	L. P. Cyl.	Large Cyl.	Large Cyl.	Large Cyl.	Large Cyl.	Large Cyl.	Large Cyl.	Large Cyl.
79.7		79.1		81.00	79.625	78.11	80.833	81.071	79.5	30.875
.4519	.735	.554	.795	.0475	.096	.155	.042	.081	.163	.437
2.69	5.097	1.74	4.24	11.82	7.62	5.32	12.62	8.57	5.11	2.18
24.39		26.5		24.00	23.78	24.22	24.66	25.285	25.52	24.00
29.15		30.34		29.83	29.83	29.83	29.83	29.83	29.83	29.83
14.31		14.89		14.64	14.64	14.64	14.64	14.64	14.64	4.64
90.00		78.00		86.3	86.75	86.00	85.00	84.66	75.25	85.00
85.05		76.66		84.66	84.00	83.33	81.00	80.33	74.75	83.00
64.66		60.00		66.00	66.00	66.00	66.00	66.00	66.00	66.00
92.53		80.66		96.66	94.25	93.00	91.00	95.00	93.75	100.50
125.26		115.33		133.66	131.50	129.33	121.33	120.66	117.00	137.50
49008.		7268.		4025.	5345.	5790.	5030.	4671.	6836.	5116.
3217.22		3634.		2236.11	2695.41	2824.39	2395.24	2774.9	3230.62	2716.94
53.62		60.56		37.2685	44.923	47.073	39.9206	46.248	53.84	45.282
4.47	7.52	72.41	13.62	74.3	76.1	72.75	74.16	75.5	76.1	26.9
8.78	22.83	87.30	28.51	88.94	90.74	87.39	88.80	90.14	90.74	41.54
4.40	12.93	47.05	16.89	10.74	12.34	15.80	10.56	11.73	16.64	16.84
7.69	6.26	42.89	4.89	8.84	8.64	9.89	7.84	7.64	9.03	8.73
9.200		14.733								
	3.253		3.317	4.560	4.400	4.400	2.960	2.774	3.680	4.720
3.0608	14.908	45.33	19.34	21.643	27.273	32.328	23.449	27.54	36.94	25.159
.75	2.25	.75	2.25	2.25	2.25	2.25	2.25	2.25	2.25	2.25
5.4372	Both. 102.263	65.911	Both. 134.53	47.24	71.75	89.141	54.838	74.618	116.015	66.74
4.4716	Both. 94.23	65.112	Both. 125.75	42.329	65.83	82.93	49.574	68.518	109.37	60.809
32278.50		5379.75		2988.75	4184.25	4782.0	2988.75	2988.75	5379.75	4184.25
1977.00		232.00					126.25	55.00	127.00	64.35
569.25		77.00		7.00	2.00	12.0	14.0	8.30	33.00	4.66
34854.75		5688.75		2995.75	4186.25	4794.0	3129.0	3052.05	5539.75	4253.28
49001.		7259.		4035.	5314.	5787.	5041.0	3831.	6870.	5090.
15.230		1.9975		1.808	1.971	2.0489	2.1046	1.698	2.142	1.873
2288.558		2847.9		1656.94	2125.0	2339.7	1486.74	1797.4	2686.25	2270.83
3.76	1485.66	2323.1	2180.61	993.364	1179.22	1546.81	900.29	1162.25	1885.67	1604.76
	.056		.040				.0405	.018	.022	.015
	.016		.010	.002	.0005	.002	.004	.0027	.006	.001
	.072		.050	.002	.0005	.002	.0445	.0207	.028	.016
.66363	.64916	.81572	.76569	.59951	.55492	.66111	.60554	.64663	.70197	.70668
3818.										
250.64										
8.84										
19.8										
201.01										
22.3798		21.169		35.075	29.616	26.247	27.113	24.088	23.154	34.026
1.852	14.528	17.268	16.209	21.023	17.755	17.352	16.418	15.576	16.253	24.045
24.287		22.647		39.144	29.280	28.21	29.99	26.23	24.56	37.3683
Actual.		Calculated.		Calculated.	Calculated.	Calculated.	Calculated.	Calculated.	Calculated.	Calculated.
2.451		2.318		3.841	3.243	2.874	2.969	2.638	2.535	3.726
2.660										
1.966										
2.133										
9.131										
9.206										
10.281										
11.385										
11.479										
12.820										

TABLE NO. 2, SHOWING THE RESULTS OF EXPERIMENTS WITH THE STEAM MACHINE

MADE AT THE U. S. NAVY YARD, BOSTON, MASS., IN AUG

		NAME OF STEAMER	"RUSH."							
		STYLE OF ENGINE	COMPOUND.							
1	Dimensions of Engines	Diameter of cylinders, inches	Small. 24		Large. 38					
2		Stroke of pistons, "	27		27					
3		Size of cylinder ports, "	18 by 1¼		2 of 27 by 1 3-16					
4		Ratio of displacement to clearance and passages	.07867		.05849					
5	Governing dimensions of Boilers (horizontal tubular type	Grate surface, square feet	57.							
6		Heating surface, " "	1572.85							
7		Cross area tubes for draft, " "	7.74							
8		Ratio grate to heating surface	27.58							
9		Ratio cross area tubes to grate surface	7.37							
10	Designation of Experiments	NUMBER FOR REFERENCE	1		2		3			
11		DESIGNATION IN LOG	Long run, Rush.		A.		B.			
12	Time	DATE OF EXPERIMENT, 1874	H. P. Eng. Aug. 3 to 6.	L. P. Eng. Aug. 3 to 6.	H. P. Eng. Aug. 6.	L. P. Eng. Aug. 6.	Aug. 17			
13		DURATION OF EXPERIMENT, hours	55		6		2.9166			
14	Steam Cut-off and Expansion	Average steam pressure in boiler, lbs.	60.06		36.731		68.70			
15		" " " " intermediate chamber, "		5.531		4.34				
16		" cut-off in fractions of stroke	.35995	.4652	.5976	.5292	.1827			
17		Ratio of expansion	2.4586	Both. 6.2157	1.5948	Both. 4.0301	4.4573			
18	Vacuum and Barometer	Average vacuum in condenser, inches	26.495		26.21		25.862			
19		" barometer, "	30.1757		30.147		30.195			
20		" " lbs.	14.8122		14.798		14.821			
21	Temperatures (Fahrenheit Scale)	Air. Average temperature external air, degrees	66.23		66.00		67.06			
22		" " engine room, "	87.86		84.23		78.428			
23		Water. " " sea water, "	59.96		59.615		66.00			
24		" " discharge water, "	90.56		93.77		99.857			
25		" " hot well, "	110.306		108.77		115.74			
26		" temp. feed water from tanks, "	114.04		114.42					
27	Revolutions	Total revolutions	233773.		19971.		9887.			
28		Average " per hour	4250.42		3328.5		3389.83			
29		" " per minute	70.8403		55.475		56.497			
30	Indicator Diagrams	Average initial pressure in cyl. above atmosphere, lbs.	67.467	8.65	35.4473	7.1842	65.613			
31		" total initial pressure in cylinder, "	82.2792	23.4622	50.2453	21.9822	80.434			
32		" " terminal " " "	29.4412	9.222	27.8506	9.1664	15.344			
33		" " cushion " " "	31.7332	11.1032	29.0618	8.048	7.508			
34		Portion stroke where cushion press. were measured	.8888	Full.	.9005	Full.	Full.			
35		Average back pressure in cylinder, lbs.	Maximum. 9.50	3.4611	Maximum. 8.2305	3.415	3.4487			
36		" mean effective pressure in cylinder, "	29.6848	12.7246	18.8818	12.2907	34.439			
37		" " total pressure in cylinder, "	39.1848	16.1857	27.1123	15.7147	37.8877			
38		Estimated friction pressure, "	2.5	1.5	2.5	1.5	3.00			
39	Power	Indicated horse-power (effective)	127.918	Both. 296 547	63.717	Both. 168.652	185.872			
40		Net horse-power	117.145	Both. 239.432	53.0367	Both. 145.1745	160.680			
41		Total horse-power	127.918	Both. 304.254	63.717	Both. 197.787	204.485			

[CONCLUDED ON NEXT PAGE.]

ACHINERY OF THE U. S. REVENUE STEAMERS "RUSH," "DEXTER," AND "DALLAS,"

N AUGUST 1874. (FOR FURTHER PARTICULARS SEE TEXT.)

"DEXTER."							"DALLAS."				
HIGH PRESSURE, CONDENSING.							LOW PRESSURE, CONDENSING.				
			26						36		
			36						30		
			16 by 2						33 by 2¼		
			.0537						.0802		
			57						57		
			1572.85						1689.24		
			7.74						7.84		
			27.58						49.63		
			7.37						7.27		
3	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**
B.	C.	Long run, Dexter.	D.	G.	F.	E.	L.	K.	Long run Dallas.	J.	H.
Aug. 17	Aug. 17.	Aug. 14-16.	Aug. 17.	Aug. 17.	Aug. 17.	Aug. 17,	Aug. 23.	Aug. 23	Aug. 20-21.	Aug. 23.	Aug. 23.
2.9166	1.4166	34.5	0.650	1.3166	1.200	0.9166	1.5166	1.550	31.	1.60	1.5333
68.70	69.286	67.12	66.42	40.625	39.9	41.875	35.40	35.286	31.96	33.7	27.40
.1827	.2335	.2483	.3331	.262	.38114	.4518	.13296	.19723	.26445	.28774	.38585
4.4573	3.6088	3.489	2.7239	3.3377	2.4232	2.0845	5.0674	3.8936	3.1341	2.9358	2.3176
25.862	25.187	25.45	25.310	26.10	26.00	25.541	26.08	26.00	25.20	25.363	24.790
30.195	30.195	30.143	30.195	30.195	30.195	30.195	30.080	30.080	29.997	30.080	30.080
14.821	14.821	14.796	14.821	14.821	14.821	14.821	14.765	14.765	14.7245	14.765	14.765
67.66	64.2	63.37	66.285	66.33	67.00	67.00	69.75	72.00	74.35	72.66	69.833
78.428	79.6	82.27	85.428	86.00	86.00	85.66	75.25	76.80	88.20	76.83	73.166
66.00	65.00	63.66	65.859	67.66	66.66	67.00	67.75	68.00	68.75	66.00	67.666
99.857	104.8	102.45	100.14	99.66	95.66	99.00	103.50	100.00	98.93	103.50	108.66
115.74	121.	113.60	119.28	107.33	108.66	117.33	122.00	122.00	128.41	128.66	134.00
		113.65							122.76		
3887.	5466.	126403.	2840.	4017.00	3978.	3338.	4430.	5294.	114426.	6190,	5839.
3389.83	3858.35	3663.85	4369.23	3050.88	3315.00	3641.45	2920.88	3415.48	3691.16	3868.75	3808.04
56.497	64.3059	61.0642	72.8205	50.8481	55.25	60.6909	48.6813	56.9248	61.5193	64.4791	63.4674
65.613	64.500	64.4095	62.155	37.50	36.55	38.812	32.14	32.275	31.8587	31.095	24.666
80.434	79.321	79.2055	76.976	52.321	51.371	53.033	46.905	47.040	46.5832	45.860	39.431
15.344	17.642	16.8746	21.633	13.134	15.771	19.008	9.475	11.869	12.6810	14.008	14.7783
7.503	8.536	8.267	10.321	6.258	8.771	9.196	5.065	5.194	6.1505	5.875	6.182
Full.....	Full.....	Full.....	Full.....	Full.....	Full.....	Full.....	Full.....	Full.....	Full.....	Full.....	Full.....
3.4487	3.7317	3.6550	5.2710	3.1570	3.6947	4.3137	2.9730	3.3678	3.9073	4.1101	4.0911
34.439	37.1274	37.5376	42.0285	25.5826	30.6649	33.8385	18.5215	21.4471	23.5255	24.6107	24.1264
37.8877	40.8591	41.1926	47.2995	28.7396	34.2996	38.1522	21.4945	24.8149	27.4328	28.7208	28.2175
3.00	3.00	3.00	3.00	3.00	3.00	3.00	2.5	2.5	2.5	2.5	2.5
185.872	228.077	218.972	292.370	124.267	161.848	196.187	137.962	186.805	221.447	242.807	234.295
169.680	205.348	201.472	271.501	109.694	146.015	178.794	119.340	165.080	197.915	218.137	210.017
204.485	251.001	240.293	320.038	139.602	181.032	221.197	160.107	216.139	258.228	283.357	274.024

TABLE NO. 2, SHOWING THE RESULTS OF EXPERIMENTS WITH THE S

			NAME OF STEAMER			"RUSH."				
			STYLE OF ENGINE			COMPOUND.				
10	Designation of Experiments		NUMBER FOR REFERENCE			1		2		3
11			DESIGNATION IN LOG			Long run, Rush.		A.		B.
42	Water	Indicated.	Total weight feed water measured			266967.		21205.35		12944.05
43			Revolutions using same			231571.		18942.		9895.00
44			Time using same			54.482		5.69085		2.919
45			Water per hour per tank measurement			4900.108		3726.24		4434.373
46			" " " " indicator diagrams			4568.231	3604.00	3328.51	2857.56	3018.794
47			Proportion of water actually used accounted for by indicator			.9323	.7355	.8933	.7669	.6808
48	Coal and Refuse		Total coal consumed			35700.				
49			Average coal consumed per hour			649.091				
50			Coal consumed per square foot of grate per hour			11.388				
51			Percentage refuse from coal			20.978				
52			Combustible per hour			512.925				
53	Performance of Engine	Water.	Water per indicated h. p. per hour by tanks			18.3836		22.0943		23.8572
54			" " " " " " by indicator			17.138	13.521	19.736	16.9435	16.2278
55			" " net " " "			20.4656		25.6673		26.1337
56			" " total " " "			16.1052		18.8397		21.6856
57	Performance of Engine and Boiler	Coal.	Coal per indicated horse-power per hour			2.4352				
58			" " net " " "			2.7110				
59			" " total " " "			2.1334				
60		Combustible.	Combustible per indicated horse-pow'r per hour			1.9243				
61			" " net " " "			2.1423				
62			" " total " " "			1.6858				
63	Performance of Boiler	Coal.	Water evaporated per lb. coal at observed pressure and temperature			7.5492				
64			Equivalent evaporation from atmospheric pressure and temperature, 100°			7.6712				
65			Equivalent evaporation from atmospheric pressure and temperature, 212°			8.5675				
66		Combustible.	Water evaporated per lb. combustible at observed pressure and temperature			9.5533				
67			Equivalent evaporation from atmospheric pressure and temperature, 100°			9.7076				
68			Equivalent evaporation from atmospheric pressure and temperature, 212°			10.8490				
69	Calculated maximum performances based on water actually used in the different engines, but calculated for boilers of different proportions using different kinds of fuel		Coal per indicated horse-power per hour, with boilers proportioned to evaporate 9 lbs. water per lb. of coal, at pressure and temperature actually used (probably the maximum which can be obtained at sea, burning anthracite coal)			2.0426				
70			Coal per indicated horse-power per hour, using slow combustion in best land boilers, or best Welch coal in marine boilers, so proportioned that there will be evaporated 10 lbs. of water per lb. of coal at pressure and temperature actually employed			1.8384				
71	14 Steam and expansion (repeated)		Average steam pressure in boiler			69.06		36.731		68.70
72	17		Ratio of expansion			6.2157		4.0901		4.4573
73	Relative performances		By comparison of combustible consumed			1.				
74			By Comparison of water used per tank measurement			1.		1.3018		1.2977
75						.7690		.9243		.9980
76						.7706		.9261		1.
77						.6823		.8200		.8855
78						.8320		1.		1.0798
79						.6889		.8279		.8940
80						.6383		.7671		.8283

"DEXTER."							"DALLAS."				
HIGH PRESSURE. CONDENSING.							LOW PRESSURE, CONDENSING.				
3	4	5	6	7	8	9	10	11	12	13	14
B.	C.	Long run, Dexter.	D.	G.	F.	E.	L.	K.	Long run allas.	J.	H.
944.05	7831.95	178867.	4477.31	4660.93	5605.7	5745.52	5592.79	7829.9	183893.	11166.02	11149.61
895.00	5493.00	125197.	2752.00	3973.00	3968.00	3355.00	4437.	5310.	113759.	6156.	5847.
2.919	1.4236	34.1708	0.6298	1.3022	1.1909	.9213	1.5191	1.5547	30.8192	1.59121	1.5354
434.373	5501.26	5234.505	7108.44	3579.15	4683.19	6235.93	3081.74	5036.83	5966.83	7017.80	7261.54
018.794	3932.24	3576.088	5420.34	2341.96	3019.52	3088.28	2648.88	3871.18	4442.12	5148.68	5336.16
.6808	.7148	.6832	.7625	.6543	.6448	.6396	.7195	.7687	.7445	.7337	.7348
.....	.....	23655.5	.....	.....	.....	.....	.....	.....	23523.75	.....	.....
.....	.....	685.667	.....	.....	.....	.....	.....	.....	758.831	.....	.....
.....	.....	12.029	.....	.....	.....	.....	.....	.....	13.313	.....	.....
.....	.....	20.291	.....	.....	.....	.....	.....	.....	20.503	.....	.....
.....	.....	546.538	.....	.....	.....	.....	.....	.....	603.247	.....	.....
23.8572	24.1202	23.905	24.3131	28.802	28.9356	31.7864	26.6866	26.9603	26.9447	28.9007	30.9932
16.2278	17.2408	16.3308	18.5390	18.8462	18.6564	20.3290	19.2000	20.7230	20.0595	21.2048	22.7753
26.1337	26.2405	25.9813	26.1820	32.6284	32.0734	34.8786	30.8508	30.5176	30.1484	32.1693	34.576
21.6856	21.9178	21.784	21.6037	25.6383	25.8694	28.1924	22.9955	23.3014	23.1069	24.7649	26.4996
.....	.....	3.1313	.....	.....	.....	.....	.....	.....	3.4267	.....	.....
.....	.....	3.4033	.....	.....	.....	.....	.....	.....	3.8341	.....	.....
.....	.....	2.8534	.....	.....	.....	.....	.....	.....	2.9386	.....	.....
.....	.....	2.4959	.....	.....	.....	.....	.....	.....	2.7241	.....	.....
.....	.....	2.7127	.....	.....	.....	.....	.....	.....	3.0480	.....	.....
.....	.....	2.2744	.....	.....	.....	.....	.....	.....	2.3361	.....	.....
.....	.....	7.6342	.....	.....	.....	.....	.....	.....	7.8632	.....	.....
.....	.....	7.7789	.....	.....	.....	.....	.....	.....	7.8458	.....	.....
.....	.....	8.6878	.....	.....	.....	.....	.....	.....	8.7625	.....	.....
.....	.....	9.5776	.....	.....	.....	.....	.....	.....	9.8912	.....	.....
.....	.....	9.7591	.....	.....	.....	.....	.....	.....	9.8693	.....	.....
.....	.....	10.8994	.....	.....	.....	.....	.....	.....	11.0225	.....	.....
.....	.....	2.656	.....	.....	.....	.....	.....	.....	2.994	.....	.....
.....	.....	2.3905	.....	.....	.....	.....	.....	.....	2.694	.....	.....
68.70	69.286	67.12	66.42	40.625	39.9	41.875	35.40	35.286	31.96	33.7	27.40
4.4573	3.6688	3.489	2.7239	3.3377	2.4232	2.0845	5.0674	3.8936	3.1341	2.9358	2.3176
.....	.....	1.2970	.....	.....	.....	.....	.....	.....	1.4156	.....	.....
1.2977	1.3120	1.3003	1.3225	1.5667	1.5739	1.7291	1.4516	1.4665	1.4657	1.5721	1.6860
.9980	1.0090	1.	1.0171	1.2048	1.2104	1.3297	1.1164	1.1278	1.1272	1.2090	1.2965
1.	1.0110	1.0020	1.0191	1.2073	1.2129	1.3324	1.1186	1.1301	1.1294	1.2114	1.2991
.8855	.8952	.8872	.9023	1.0089	1.0739	1.1797	.9904	1.0006	1.	1.0726	1.1503
1.0798	1.0917	1.0819	1.1004	1.3036	1.3096	1.4387	1.2078	1.2202	1.2196	1.3081	1.4028
.8940	.9038	.8958	.9110	1.0793	1.0843	1.1911	1.	1.0102	1.0097	1.0805	1.1614
.8283	.8374	.8300	.8441	1.	1.0046	1.1037	.9265	.9360	.9355	1.0084	1.0761

"The relative performances shown decimally in lines 73 to 80, inclusive, with different experiments as unity, will be found convenient for comparison."

Annexed will be found specimens of the indicator diagrams taken during the principal runs of both series of trials.

Discussion of the Results.—The foregoing statement necessarily involves so many details that we add, as convenient for reference from time to time as we proceed with the discussion, a recapitulation of the distinguishing features of the engines tested, and of the general scope of the two series of investigations.

1. The Coast Survey steamer Bache was provided with a compound engine of the steepled type (that is the small cylinder was arranged above the other, and the pistons had a common rod). The larger cylinder was steam jacketed, and so arranged that it could be operated independently, using steam of the same pressure and with the same degree of expansion as when both cylinders were working together as a compound engine. Trials were made of the two systems of working, both with the steam-jacket in use and when same was disconnected, and the amount of water collected from the jackets and intermediate chamber was separately weighed and noted. All the experiments, except one, were made with an approximate steam pressure of 80 pounds, and with different degrees of expansion for each system of working, with and without use of jacket. The indicated power was measured, also the cost of the same in steam (shown by the weight of feed-water used). The evaporative efficiency of the boiler was also determined.

2. One of the revenue steamers, the Rush, was provided with a compound engine, constructed on the "fore and aft" system (that is the cylinders were at the same level, and in this case the pistons were connected to cranks at right angles). Both cylinders were steam-jacketed. The other two revenue steamers, viz., the Dexter and Dallas, had non-compound engines, with un-jacketed cylinders. The compound engine of the Rush was operated in two different runs at the approximate steam pressures of 70 and 40 pounds. The single engine of the Dexter was operated with the same steam pressures and at different degrees of expansion for each pressure. The engine of the Dallas was operated at an approximate steam pressure of 35 pounds, and at different degrees of expansion. The boilers were all substantially similar. The performance was obtained by measuring the indicated power and its cost in steam, as shown by the weight of feed-water

used. The evaporative efficiency of the boilers was also ascertained during the longer experiments.

From the two series of experiments may be gathered the following information, viz. :

1. The saving by the use of a steam-jacket on the cylinder of a non-compound engine, and the larger cylinder of a compound engine.

2. The relative saving that may be obtained by the use of a compound engine, as compared with a single engine, operated at the same or a different steam pressure, or at the same or a different degree of expansion.

3. The probable value of a steam jacket on the smaller or high-pressure cylinder of a compound engine.

4. The influence which the size of a steam cylinder has upon the economy of fuel.

5. The relative cost of the power, at different steam pressures, in compound and non-compound engines.

6. The most economical point of cut-off for the steam pressures employed.*

1. The Advantages of the Steam Jacket—(1 A). Referring to Table No. 1, and comparing the minimum costs for each method of working, we find that the single cylinder of the Bache when operated without the steam-jacket required (Exp. 13, line 46), 26.247 pounds of feed water per indicated horse-power per hour, and that with steam-jacket in use there was required (Exp. 16, line 46) but 23.154 pounds, showing that the saving by the use of the steam-jacket on a single cylinder engine worked at its most economical point of cut-off is 11.78 per cent. With more expansion, as shown by comparing the previous experiments for each method of working the jacket produces a greater saving, but the steam is in all cases being cut off too short for maximum economy, as will be discussed hereafter.

(1 B.) When the engine of the Bache was operated as a compound engine, with steam-jacket not in use, experiment 2 shows a cost of 23.036 pounds of water per I. H. P. per hour, and experiment 6 with steam-jacket in operation, a cost of 20.332 pounds. The saving in steam by the use of the jacket on the larger cylinder of a compound engine is then shown to be 11.73 per cent.

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* The above subjects are discussed in the order named, and paragraphs to which it may be desirable to refer, are designated by letters affixed to the numbers referring to the subjects.

2. Saving by use of Compound Engine.—(2 A). The minimum cost of the power with the single cylinder of the Bache and steam-jacket in use, experiment 16, is 23.154 pounds, and with engine compounded and steam-jacket on large cylinder in use, experiment 6, it is 20.332 pounds, so the compound engine was operated with a saving of 12.19 per cent. in this case, as compared with the single engine.

(2 B.) The above experiments were made at the same steam pressure, but with a less degree of expansion in the single engine. the steam being expanded nearly seven times (6.975) in the compound engine, and but little more than five (5.11) times in the single engine. With the steam expanded eight and one-half times (8.57) in the single engine (Exp. 15), the cost is 24.088 pounds using the same steam pressure, so the compound engine shows a saving compared therewith of 15.6 per cent. The difference increases as the expansion is increased in the single engine.

(2 C.) The minimum cost of the power with the single cylinder and steam-jacket not in use, experiment 13, is 26.247 pounds, and with engine compounded, without steam-jacket, experiment 2, it is 23.036 pounds, so without using steam-jackets in either case, the compound engine operated with a saving of 12.23 per cent. as compared with the single engine.

(2 D.) With steam-jacket in use on larger cylinder of compound engine, experiment 6, and not in use on single engine, experiment 13, the costs as before stated, were respectively 20.332 and 26.247 pounds, showing a saving by the use of the former under conditions stated of 22.54 per cent.

(2 E.) In the experiments with the revenue steamers it will be seen (Table No. 2, line 76, Exp. 1 and 3) that the relative costs of the power in the compound engine of the Rush with both cylinders jacketed, and in the single engine of the Dexter, with un-jacketed cylinder, were as 0.7706 to 1, corresponding to a saving by the use of the compound engine with both cylinders jacketed, as compared with an engine with single un-jacketed cylinder, of 22.94 per cent., or practically the same as shown on the Bache with only the larger cylinder of the compound engine jacketed.

(2 F.) Assuming that a steam-jacket on the single cylinder of the Dexter would have reduced the cost in the same proportion that it did in the Bache, viz., 11.78 per cent., the cost of the power in the single cylinder engine which was 29.77 per cent. greater than in the compound engine (Table No. 2. line 74, Exp. 1 and 3) would have been reduced

(12.977 × 11.78=) 15.29 per cent., and the relative costs would have been as 1 to 1.1448, equivalent to a saving of (1.1448—1 × 100 ÷ 1.1448 =) 12.65 per cent., by the use of the compound engine with jacketed cylinder, as compared with the single engine with jacketed cylinder.

As above stated (2 A), the experiment with the Bache showed a saving of 12.19 per cent. under similar conditions.

The experiments do not furnish conclusive information as to what the relative performances of compound and non-compound engines of larger sizes would be. It seems probable, however, that in such case the compound engine would show still greater advantages.

In the revenue experiments above cited (2 E), the saving of 22.94 per cent. was reduced to 12.65 per cent. (2 F), by assuming that a steam-jacket on the cylinder of a single engine would save as much as it did on the Bache which is not probable, for the reason that the cylinder of the Dexter was larger than that of the Bache, and it is an evident fact that the ratio of capacity to jacket surface decreases as the size of the cylinder is increased.*

This reasoning does not apply to the experiments made on the Bache with the same engine operated on both systems, but in that case the compound engine was not constructed for maximum economy, while the cylinder used for the single engine was probably as good as could be made. The latter was thoroughly steam-jacketed at sides and in bottom and cover. It also had large cylinder ports, and the minimum amount of space in clearances and passages. Tight pistons were used in all cases.†

When the engine of the Bache was used as a compound engine, the upper cylinder, though well felted and lagged, was necessarily exposed to more refrigerating influence than in compound engines on the "fore

* To settle the question as to the economy of the steam-jacket on a single engine of practically the same size as the compound engine of the Rush, arrangements are being made for a series of trials on a revenue steamer recently completed, the "Gallatin" which has machinery adapted for the purpose.

† It will be interesting to add that the direct steam connection for large cylinder of the Bache was originally designed by the writer as an auxiliary arrangment to be used in case of accident to the other cylinder, and the drawing provided therefor, a pipe of the same size as that for the upper cylinder. The contractors, Messrs. Pusey, Jones & Co., of Wilmington, Del., voluntarily increased the size of this pipe so that the two systems could be fairly tested in the same apparatus, though the duties of the vessel were such that it was not convenient to make the trial till about 3½ years after. This same firm, shortly after the construction of the Bache, built also the first of the long stroke high-pressure engines which revived the interest in that system, and I observe that they applied the steam-jacket to the cylinder and connected the cylinder to frames, through legs cast on former to hinder the transmission of heat to latter, the same as was provided for in drawings furnished for the engine of the Bache.

and aft" system. The omission of the steam-jacket on the small cylinder may also have occasioned some slight loss, which subject is discussed hereafter. These disadvantages were considered in the first instance as of less consequence than the saving of space, &c., accomplished by adopting the system in that particular location, but in making a comparison of compound and non-compound engines, it is proper that they should be considered,

Both series of experiments appear to show then that the compound engines were at least not tried at any advantage as compared with the single engines; on the contrary, the indications are that still greater comparative economy would be shown by the compound system in larger engines.

3. VALUE OF SMALL CYLINDER JACKET.—These experiments appear also to corroborate the views held by the writer at the time (1870), the engine of the Bache was constructed viz.: that the steam-jacket on the smaller or high pressure cylinder of a compound engine, working with the ordinary degree of expansion, was, contrary to the views of Rankine on the subject,* of comparatively little value.

(3 A.) The experiments with the compound engine of the Rush, where both cylinders were jacketed, compared with the single engine of the Dexter with un-jacketed cylinder showed a saving (see 2 E) of 22.94 per cent., and in experiments on the Bache, the compound engine with jacket on large cylinder showed a saving, compared with single cylinder used without jacket (see 2 D) of 22.54 per cent., so the additional jacket on the small cylinder of the Rush did not sensibly affect the comparison on this basis.

(3 B.) On the Bache we find that the saving by the use of a compound engine with large cylinder jacketed, as compared with a single cylinder used with jacket, is (see 2 A.) 12.19 per cent., and by indirectly comparing the performance of the Bache and the revenue steamers, the compound engine of the Rush with both cylinders jacketed, shows a saving (see 2 F) of 12.65 per cent. The saving by the jacket on the small cylinder could not then be more than (12.65–12.19=) 0.46 per cent. It is probable, however, that in compound engines constructed on the "fore and aft" system, the steam-jackets on both cylinders heat the intermediate steam as it passes from one cylinder to the other, and thereby reduce the cost of the power.

* Rankine on "The Steam Engine," Art. 286.

(3 C.) The opinion as to the relative value of the steam-jackets on the two cylinders of a compound engine was founded upon the views of the writer expressed in print as early as 1866-7, which were in substance, that the great difference between the theoretical and practical performances of the steam-engine could be satisfactorily accounted for by the differences of temperature to which the interior surfaces of the cylinder are practically subjected. The metal of the cylinder is cooled by the exhaust steam and must be reheated during the next stroke, which causes condensation of part of the incoming steam; the resulting water is re-evaporated, partially during the expansive portion of the steam stroke but mostly during the next exhaust stroke, thereby cooling the cylinder again, and the result is to transfer heat (by the alternate condensation and re-evaporation described) directly to waste in the atmosphere or condenser.*

A full discussion of this branch of the subject would occupy too much space in this paper. It was considered that the range of temperature in the smaller cylinder of a compound engine is generally less than in the large cylinder and, moreover, that any heat transferred (so to speak) past the pistons of the smaller engine would do useful work in the second cylinder. These views are apparently sustained by the experiments, but it should not be assumed that the jacket is of less value simply because high-pressure steam is used in the small cylinder, and that, therefore, a jacket is unnecessary on a high-pressure condensing engine. Quite the contrary is true, for in such case the interior surfaces of the cylinder are

* It had previously been suggested that a portion of the loss could be accounted for in the manner indicated. The writer, after reading the account of Tyndall's experiments, showing the facility with which aqueous vapor radiated and absorbed radiant heat, and finding by calculation that it was necessary to heat and cool the metal of a cylinder during each stroke, but a very small distance below the surface to occasion all the loss observed, became convinced that nearly all the loss could be accounted for in this way, and it occurred to him that if the interior walls of the cylinder were made of non-conducting material, the loss would be greatly reduced. Accordingly experiments were made by the writer in 1866, which have been referred to in several publications. The following brief description appeared in an article on Compound Engines in the American Artizan of March 8th, 1871.

"The nature of the loss was proved in the following manner: I constructed two cylinders of like dimensions, one of glass, the other of iron, in such a manner that either could be attached to a valve which regularly admitted steam from a boiler to the cylinder and permitted its exhaust into a condensing coil lying in a tub of water."

"The capacities of the two cylinders were made exactly the same, as was shown by transferring water from one to the other. When put in turn in the condition of a steam-engine cylinder, the iron cylinder used (averaging the experiments) fully twice as much steam as the glass one, shown by the fact that twice the quantity of water came through the condensing coil for the same number of movements of the valve. Steam of the same pressure was used in both cylinders, and the experiments were many times repeated with substantially the same results."

exposed to a variation of temperature equal to that in both the cylinders of a compound engine, and the jacket becomes of the greatest importance.

(3 D.) It is probable that the steam-jacket produces economy by drying and superheating the steam near the heated surfaces. If this be done promptly the heat imparted to the steam assists in performing useful work during the expansive portion of the stroke. Even during the exhaust stroke the dry steam near the surfaces of the cylinder will absorb very little heat compared to that required to evaporate particles of water which are always present when no jacket is used. The dry steam can only absorb heat at a rate proportioned to its specific heat, which is less than half that of water. The watery particles will take up both sensible and latent heat, or, for equal weights under actual pressures used, about two thousand times as many heat units as the dry steam. Could the steam in a cylinder be discharged simply saturated or slightly superheated, either by previous superheating or the use of efficient steam-jackets, the loss would be very small. At high expansions this is impracticable. Some water is always present and in its evaporation cools the metal surfaces somewhat, and it is always better to resupply the heat from the steam-jacket than by condensation of incoming steam, as in the latter case the resulting water of condensation remains in the cylinder and causes increased losses in manner previously indicated.

From these considerations it may be inferred that steam-jacketed cylinders would be most efficient of comparatively small diameter and long stroke, to obtain as much surface in proportion to volume as possible, and that for unjacketed cylinders the surface in relation to volume should be reduced as much as possible.

4. Economy of Steam as Influenced by the Size of the Cylinder.—It is a well known fact that large engines are more economical per unit of power furnished than small ones. It is related that Watt was led to his invention of a separate condenser by observing the excessive quantity of steam required to operate a small model engine as compared with that found sufficient for engines of practical sizes—the vacuum being produced in both by admitting the condensing water directly into the steam cylinder. We have previously called attention to the fact that Watt, in producing condensation in a separate vessel, only partially overcame the difficulty; steam chilled by the performance of work being such an excellent radiator and absorbent of radiant heat as to cool the interior surfaces of a cylinder, upon reduction of pressure by condensa-

tion, even in a separate vessel, to a very material degree, if not to the same extent as if the water were directly admitted to the cylinder.*

The manner in which the loss takes place was discussed under the previous heading, and it is evident that the amount of heat transferred to waste in an unjacketed cylinder will, for a given range of temperature, vary as the amount of metal surface and inversely as the volume of steam exposed to the same. As the size of the cylinder is increased the volume increases in a more rapid ratio than the enclosing surfaces, hence the loss, relative to the unit of volume and power, decreases as the size of the cylinder is increased.†

(4 A.) The minimum cost of the power, using a single engine without a steam-jacket was on the Bache (Table No. 1, Exp. 13, line 46) 26.247 pounds of feed-water per horse-power per hour, and on the Dexter (Table No. 2, Exp. 3, line 53) it was 23.857. Again, on the Bache, using compound engine and jacket on the larger cylinder, the cost was (Table No. 1, Exp. 6, line 46) 20.332 pounds and on the Rush (Table No. 2, Exp. 1, line 53) it was 18.384 pounds. The engine of the Bache was smaller than that of either of the other steamers, and, as shown, the cost was in both cases considerably more. We have already pointed out that the relative performances for different methods of working in one series correspond well with those shown by the other, such, for instance, as the saving of the use of the steam-jacket and the relative economy due to a compound engine, but we now find that the actual costs shown in one series cannot be directly compared with those obtained in the other.

It may be accepted as a general fact that an experiment made with one engine cannot be used directly as a basis of comparison except for engines of similar size operated under similar conditions. The want of general information on this subject has often caused great misapprehension. Engineers have again and again made improper comparisons and often have conscientiously drawn directly opposite conclusions from the same data, when neither side had access to information sufficiently complete to refute the arguments of the other.

(4 B.) Referring to the experiments under discussion we find that the minimum cost of the power with the jacketed-cylinder of the

* The subject was treated in this manner in a lecture delivered at Sheffield Scientific School in the winter of 1870-71, which never having been completely written out, was not published. See reference to Tyndall experiments with radiant heat in preceding foot-note.

† This statement has been demonstrated by numerous experiments, particularly those referred to in the foot-note on the first page of the discussion, and may be confirmed by comparing the results of similar experiments in the two series herein discussed.

Bache, non-compound (Table No. 1, Exp. 16, line 46) was 23.15 pounds of water per horse-power per hour, and that with the unjacketed cylinder of the Dexter (Table No. 2, Exp. 3, line 53) it was but 23.86 pounds. Had there been but these two experiments to compare, many would naturally have held that there was little or no economy in the steam-jacket, and it is only by having the extended series of experiments on the Bache with and without the use of jackets on the same cylinder, that we are enabled to prove that there is economy in the use of the jackets; and by comparing experiments with engines of different sizes to show further that the actual performances of different engines are not directly comparable, while the relative performances for each engine, operated under different conditions, are properly comparable with similar relative performances of other engines.

(4 C.) It should be here observed that in the above comparison of the results of experiments with the steamer Bache and of the revenue steamers, the former shows more inferiority than can be attributed simply to the difference in size, and we are of the opinion that it was due somewhat to the quality of the steam furnished by the boilers. The boiler of the Bache was constructed to give a high evaporation, and the combustion was so slow that the steam in the steam-chimney received practically little heat from the escaping gases. On the revenue steamers, the boilers were made to develop the maximum power for a given space —the tubes were shorter, the draft freer, and the experiments being tried at maximum power, the gases passed through the steam chimney at a higher temperature than on the Bache, so that the steam was more thoroughly dried. It appears incidentally, then, that a boiler with a less evaporative power than another, may, within certain limits, furnish steam of a better quality, and thereby produce increased economy in the engine. This is not fully demonstrated by these experiments, as both the size of the engines and the proportions of the boilers are different in the two series, but that there is some such compensation appears probable.*

In these experiments, however, the higher evaporation in the boiler of the Bache, more than compensated for the loss of efficiency in the engine, the best results with the compound engine of the Bache and that of the Rush being as follows:

* It is to be regretted that the quality of the steam was not tested in all the experiments by means of a calorimeter. The writer, previous to the trial of the Bache, developed and put in practice a very simple apparatus for the purpose, but it was thought that there was so much to be done in a limited time, that the solution of the more important problems could only be attempted.

	Bache.	Rush.
	Table No. 1, Exp. 6.	Table No. 2, Exp. 1.
Water actually used per I.H.P., per hour....	(line 46) 20.332	(line 59) 18.384
Water actually evaporated per pd. of coal....	(line 55) 9.131	(line 63) 7.549
Coal consumed per I.H.P., per hour.....	(line 49) 2.227	(line 57) 2.435

The presentation shows that while there may be an interesting subject (which we shall investigate further), as to the compensation attainable in the engine by utilizing the waste heat of a boiler with low evaporative power, the higher evaporation will probably, as in this case, prove the better practically. Referring to Table 2, Exp. 1, line 70, it will be seen that with an evaporation of ten pounds of water per pound of coal, (a result attainable under the conditions named), one horse-power, would have cost but 1.838 pounds of coal per hour.

5. Economy of Steam as Influenced by the Steam Pressures employed.—The investigations with which we have been associated show invariably that, other things being equal, the higher the steam pressure the greater the economy. The saving, however, decreases rapidly, using ordinary engines, after a pressure of eighty pounds per square inch is reached, so much in fact that it is doubtful if pressures in excess of one hundred pounds would give a sufficient economy of fuel to counterbalance the extra expense in constructing and maintaining the boilers. We have little information as to what can be done at pressures above one hundred pounds, with engines particularly designed for the purpose, and it is probable that a saving of space occupied by the machinery might in some cases warrant the use of very high pressures even with ordinary engines. Within the limits of common practice, the saving by the use of the higher pressures is very important, and some valuable information on the subject may be obtained from the experiments under discussion. We first examine the results due to using different steam pressures in the same cylinder.

(5 A.) With the non-compound engine of the Dexter, we find (Table No. 2, line 76) comparing experiments 3 and 7, which show the minimum cost at the approximate steam pressures respectively of 70 and 40 pounds, that the power at latter pressure cost 20.73 per cent. more than at the former. This is at about the same degree of expansion, and therefore doing less work, with the less pressure. Were it necessary to do the

same work, in the same cylinder, as is the case in practice, comparing runs 3 and 9, we find the power would cost 33.24 per cent. more.

(5 B.) Experiments 13 and 17 on the Bache (Table 1), made at the steam pressures of 78 and 31 pounds, respectively, show a cost for the latter 29.6 per cent. greater than for the former.

(5 C.) In the case of the compound engine of the Rush (experiments 1 and 2, Table 2), made with the steam pressures of 69 and 37 pounds, respectively, the cost of the power at latter pressure is 20.18 per cent. greater than in the former.

(5 D.) The results due to working steam of different pressures in engines properly proportioned to give the maximum economy for the pressure used, involves questions discussed in the next title. Referring, however, to the results shown above, (5 A) and (5 B), it may be observed that if the lower pressure of steam is to be used, it can better be done in a cylinder proportioned as above indicated, and such was very nearly the case in the Dallas. Comparing experiments 11 and 3 (Table No. 2), where the power is nearly the same, we find (line 76) that the power in the low pressure engines of the Dallas cost 13.01 per cent. more than in the high pressure condensing engine of the Dexter.

(5 E.) Above are shown, (5 A), (5 B) and (5 C) practical comparisons of the results due to reducing the steam pressure in the same engine, which furnish a basis whereby we may account for the fact that compound engines in practical use show larger relative economies, compared with simple engines, than we have ascertained by experiment. In high pressure condensing engines, the pressure for various reasons is seldom maintained regularly at the point designed. This occurs from two causes, viz., carelessness of the operating engineer and the improper adaptation, by the designing engineer, of the size of the engine to the work it has to do. The latter, when true, as is too often the case, partially excuses the fault of the former, which subject is discussed in the next title. It is also true that no matter for what pressure the engine is designed, if it be intended to be operated with considerable expansion, the engineer soon finds that his engine works smoother with a lower pressure and less expansion, and naturally thinking, as is too often the case, that his duties are sufficiently arduous, lets his pressure fall or partially closes his throttle-valve and lengthens the cut-off for the most trivial excuses, until finally, notwithstanding his education and instructions, he really believes that it is exactly as well to work that way all the time.

The general prejudice against high pressure is in his favor, and it is not uncommon to have somebody on the vessel boast that their engineer can run with less pressure than somebody else, which is accepted as a matter to be proud of instead of being worthy of condemnation. To be sure, the expense account for coal increases somewhat, but it is attributed to the falling off due to continued service. If protestations of owners that something is wrong are repeated, and direct orders given to work more expansively, the result generally proves a failure; sometimes through lack of interest, and numerous complaints about leaky boilers, &c., and at other times trials are made of higher pressures when the engine really has got out of order and no saving can be observed. It is a fact that if the pistons have become leaky, it is as economical to use a lower steam pressure as a high one. The true remedy in such case is to refit the pistons and carry the pressure designed. We have of late, by the use of special arrangements, found no difficulty in keeping pistons continuously tight at any pressure.

(5 F.) With the compound engine, however, there are fewer mechanical difficulties in working high steam, and in most cases it is difficult to keep up the speed with low steam. For instance, examining experiments 1 and 2 with the compound engine of the steamer Rush, it will be seen (Table 2, line 39) that the power at the lower pressure is much less than with the higher (168.6 to 266.5), so the engineer, with an engine of proper size, would be obliged to increase the pressure to obtain sufficient power to propel his vessel at the speed designed. The result is that the average pressure carried for compound engines, even by careless engineers, is much higher than in single cylinder engines, and increased economy due to the steam pressure is obtained, independent of that due to the difference in engines, and we may expect to find in practice, as is commonly reported, that a compound engine operates with a saving of 20 to 25 per cent. compared with single engines using the same pressure, and that even more saving may be obtained when the single engine is greatly too large for its work, as hereinafter discussed.

6. The most economical Point of Cut-off for the Pressure employed.—When it is desired to obtain a given power, using steam of a given pressure, fixing the point of cut-off fixes also the mean pressure in the cylinder, and for a given speed of revolution, the size also of the cylinder required. Our experimental researches show that the most economical grade of expansion varies for every steam pressure, and is influenced somewhat by other conditions.

Title.		Number of experiments.	Ratio of expansion	Mean effective pressure.	Cost in water per horse-p'er per hour.		Number of experiments.	Ratio of expansion	Mean effective pressure.	Cost in water per horse-p'er per hour.
Table 6 A.		Line 1.	Line 7.	Line 25.	Line 46.		Line 1.	Line 7.	Line 25.	Line 45.
Abstract from general table No. 1, U. S. Coast Survey Steamer Bache, non-compound engine, approximate steam pressure 80 pounds.	With steam jacket in use on larger cylinder,	14	12.62	23.449	27.113	Without using steam jacket.	11	11.82	21.643	35.075
		15	8.57	27.54	24.088		12	7.62	27.273	29.616
		16	5.11	36.94	23.154		13	5.32	32.328	26.247
Table 6 B.		Line 10.	Line 17.	Line 36.	Line 53.		Line 10.	Line 17.	Line 36.	Line 53.
Abstract from general table No. 2, U. S. Revenue Steamer Dexter, non-compound engine, approximate steam pressure 70 pounds.	.....	3	4.46	34.439	23.857		5	3.49	37.538	23.905
	.....	4	3.67	37.127	24.12		6	2.72	42.028	24.313
Table 6 C.		Line 10.	Line 17.	Line 36.	Line 53.		Line 10.	Line 17.	Line 36.	Line 53.
Abstract from general table No. 2.	U. S. Revenue steamer Dexter (engine non-compound) approximate steam pressure 40 pounds.	7	3.34	25.583	28.802	U. S. Revenue Steamer Dallas, engine non-compound, approximate steam pressure 35 pounds.	10	5.07	18.522	26.687
		8	2.42	30.665	28.936		11	3.89	21.447	26.96
		9	2.08	33.898	31.786		13	2.94	24.611	28.901
Table 6 D.		Line 1.	Line 7.	Line 25*	Line 46.		Line 1.	Line 7.	Line 25.	Line 46.
Abstract from general table No. 1, U. S. Coast Survey Steamer Bache, compound engine, approximate steam pressure 80 pounds.	With steam jacket in use on larger cylinder.	4	16.85	20.379	25.108	Without using steam jacket.	1	9.146	22.421	23.765
		5	9.19	27.524	20.713		2	6.658	27.599	23.036
		6	6.97	31.843	20.332		3	5.634	29.781	23.21
		7	5.73	33.484	20.365		..	....	....	....
		10	4.24	36.94	21.169		..	....	....	....

* Mean effective pressure referred to larger cylinder.

The preceding tables have been condensed from the general tables, and show the mean pressures and costs of the power at different degrees of expansion for the engines of the several steamers :

Referring to Table 6 A, it will be seen that, with the engine of the Bache, operated non-compound, using an approximate steam pressure of 80 pounds, expanded 5.11 to 12.62 times, the higher grades of expansion were attended with positive loss, and by reference to Table 6 B it will be seen that, with the single engine of the Dexter, using an approximate steam pressure of 70 pounds, expanded 2.72 to 4.46 times, there was but little difference in economy between an expansion of 3.49 times and one of 4.46 times. We may, therefore, infer that an expansion of five times, under the conditions of these trials, using 80 pounds of steam, in single engines, is as much as can be obtained economically, and that the expansion should be somewhat reduced for a pressure of 70 pounds.

Referring to Table 6 C, it will be seen that, with the single engine of the Dexter, using an approximate steam pressure of 40 pounds, expanded 2.08 to 3.34 times, (the latter expansion is the more economical) and that, with the engine of the Dallas, using an approximate steam pressure of 35 pounds, expanded 2.94 to 5.07 times, no loss, but rather a slight gain in cost of indicated power, is shown at an expansion of 5.07 times, as compared with that at 3.89 times—this engine operating, as before stated, very economically at the pressure used. The results at the two expansions last named are, however, so nearly identical, that the cost for the net power (see Table No. 2) is least for the least expansion ; considering the experiments on the Dexter and Dallas together, we may conclude that an expansion of 3½ to 4 times is the most economical degree for steam pressures of 35 to 40 pounds.

Referring to Table 6 D, it will be seen that, even with the compound engine of the Bache, operated with an approximate steam pressure of 80 pounds, expanded 4.24 to 16.85 times, a loss resulted at the extreme degree of expansion, and that an expansion of 6 to 7 times appeared to give the best results under the conditions of the trials.

Is is not practicable, with the information available (many experiments not having been put in shape for comparison), to calculate accurately the proper rates of expansion for different steam pressures, and it is probable that no fixed rule could be framed to include the modifications due to all conditions. We give the following provisional rule, with tabulated examples :

(6 E.) RULE.—To the number representing the steam pressure above the atmosphere (P) add 37; divide the sum by 22; the quotient will represent, approximately, the proper ratio of expansion (R) for that steam pressure. That is

$$R = \frac{(P + 37)}{22}$$

EXAMPLES:

Steam pressure above atmosphere = P	5	10	25	40	60	80	100
Ratio of expansion = R	1.9	2.1	2.8	3.5	4.4	5.3	6.2

It is probable that these ratios are nearly correct for single engines of large size with details of good design, too large for single engines of ordinary construction, and too small for the better class of compound engines. The rule, though provisional, is safer to follow than the uncertainties of personal opinion, and the variations of actual practice. Further information cannot vary it materially, for the economy changes very little for expansions considerably greater or less than the most economical grade. The limit of expansion for the higher pressures are apparently well defined by the experiments discussed, but there are indications that there is no loss in using somewhat higher expansions than given by rule for steam pressures of 35 to 40 pounds—of course, however, with results inferior to those obtained by using higher pressures. Further investigations are being made on this subject.

(6 F.) As a general rule, in constructing an expansive engine, too much expansion is attempted and the cylinder is made much too large for the work to be done. This is particularly true in respect to engines designed to be operated expansively with high steam pressures. As previously referred to, the designing engineer, in almost every instance, furnishes too much cylinder to work off the steam from a given boiler at the most economical degree of expansion for the pressure intended. This is one of the most important lessons to be learned from these experiments, and many others as yet unpublished. Nearly all the marine engines constructed have cylinders of sufficient size to develop the power intended with a mean pressure of 25 to 20 pounds, and even lower. These experiments show clearly that it is not economical to expand high pressure steam sufficiently to produce so low a mean pressure, and that with such large cylinders it would be nearly, if not quite as well, to reduce the steam pressure and expansion (as we have complained previously, that the working engineers are in the habit of doing, though to an unwarranted extent). The best results shown by these experiments were obtained with a mean pressure of 34 to 37 pounds, when the boiler

pressures were from 70 to 80 pounds, and, therefore, the steam cylinders of non-compound high pressure condensing engines should be not more than two-thirds the size they are usually made. Engines so proportioned would not only work with greater economy, but also with less expansion than those with larger cylinders, so that there would be a more equable pressure throughout the stroke when high steam was used, and less trouble to the engineer. Such an engine, properly constructed and operated, would probably require but about 15 per cent. more fuel than the best form of compound engine to do the same work.

(6 G.) We are now constructing a non-compound high pressure condensing engine, which is fitted with expansive gear, adjustable only between one-sixth and one-fourth the stroke. To manœuvre the vessel, the cut-off must be thrown entirely out of gear, and distribution of steam effected entirely with main valve. It is hoped that the arrangement will prevent the temptation to reduced expansion, which would take place to an important extent, even when using a cylinder of proper size for the work to be done.

Indicator Diagrams.—Annexed will be found specimens of the indicator diagrams taken during the principal runs. Diagrams for each series of experiments on the Bache have been selected with like steam pressures and traced together for facility of comparison.

Hyperbolic curves have been dotted upon one series of the diagrams in such manner as to coincide with the experimental curves near the points of suppression. When the diagrams were taken, the indicators were carefully adjusted so as to be perfectly free in their action, and in some of the diagrams vibrations of the pencil may be observed at the points of cut-off outside the true experimental lines.

The diagrams considered together show a number of differences due to the various changes of condition. As stated previously, the steam supplied by the boilers of the revenue cutters was probably superheated slightly, and the expansion curves on diagrams *F*, *G*, *H*, from their engines more nearly correspond with the Mariotte curves of expansion than the majority of those from the engine of the Bache. The boiler of the Bache was operated much below its power when furnishing steam at the high grades of expansion used (see 4 C), and as the steam room was large, the water level very steady and extra precautions had been taken to felt the boiler, steam-pipe and valves, the steam supply must have been delivered to the engine in about the same condition as if from a boiler without any superheating surface, or say very nearly saturated or containing but a

INDICATOR DIAGRAMS, U. S. COAST SURVEY STEAMER "BACHE," AND

The numerals on the diagrams refer to the nu

A.

STEAMER "BACHE"

SINGLE ENGINE, USING STEAM JACKET.

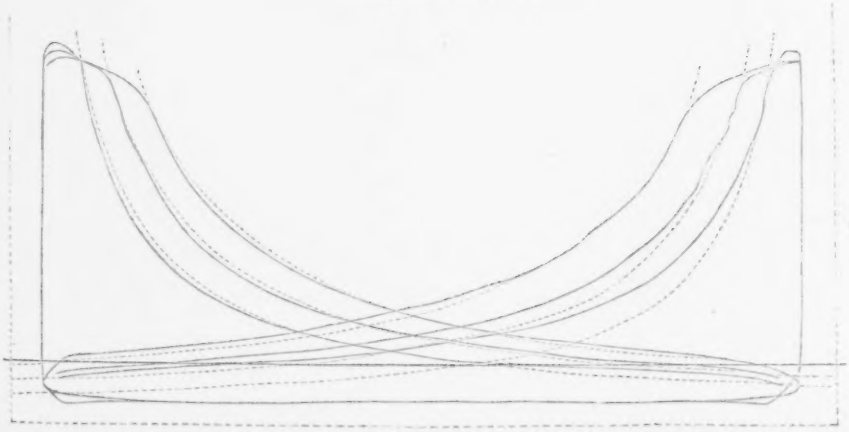
Indicator Scale, 40 lbs. per inch.

B.

STEAMER "BACHE."

SINGLE ENGINE, WITHOUT USING STEAM JACKET.

Indicator Scale, 40 lbs. per inch.



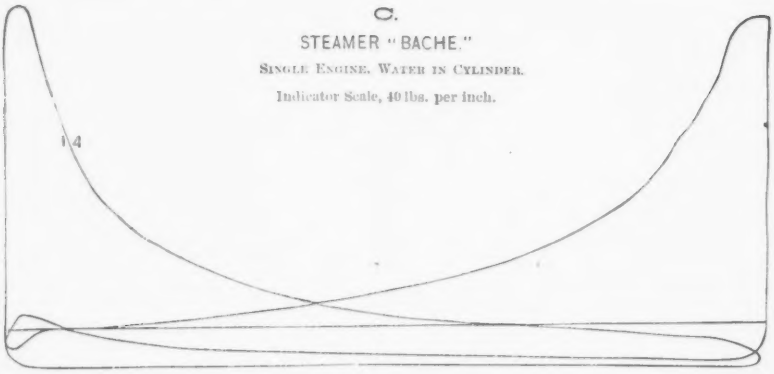
the number of the experiment in Table No. 1.

C.

STEAMER "BACHE."

SINGLE ENGINE. WATER IN CYLINDER.

Indicator Scale, 40 lbs. per inch.

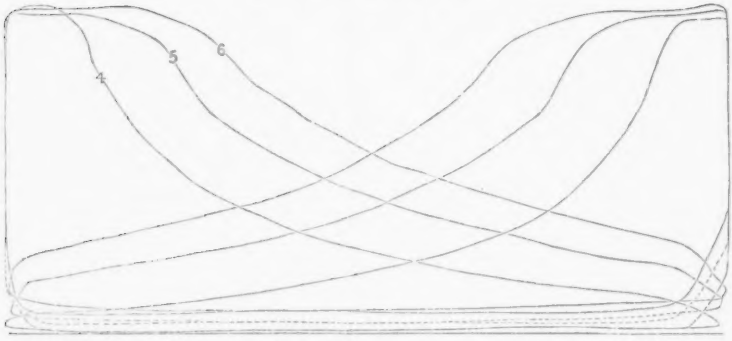


D 1.

STEAMER "BACHE."

COMPOUND ENGINE, SMALL CYLINDER.

Indicator Scale, 40 lbs. per inch.

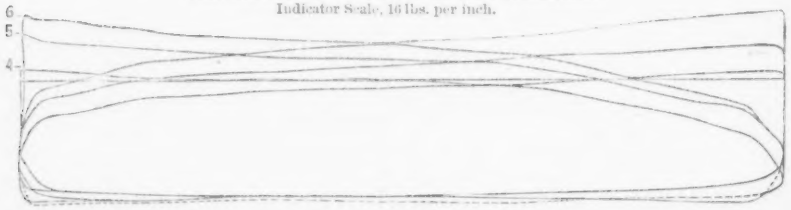


D 2.

STEAMER "BACHE."

COMPOUND ENGINE, LARGE CYLINDER. USING STEAM JACKET.

Indicator Scale, 16 lbs. per inch.



INDICATOR DIAGRAMS, U. S. COAST SURVEY STEAMER "BACHE," AND U

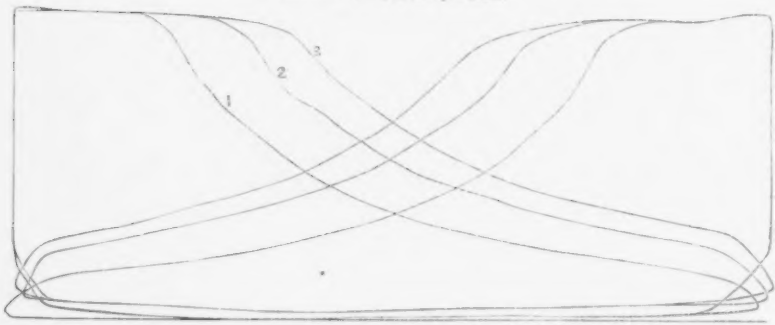
The numerals on the diagrams refer to

E 1.

STEAMER "BACHE."

COMPOUND ENGINE, SMALL CYLINDER.

Indicator Scale, 40 lbs. per inch.

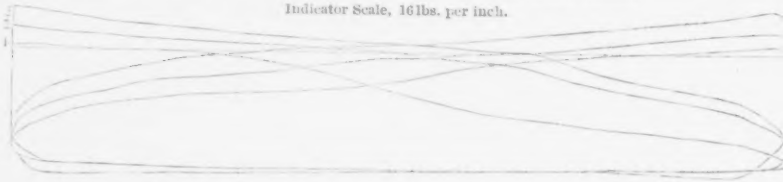


E 2.

STEAMER "BACHE."

COMPOUND ENGINE, LARGE CYLINDER, WITHOUT USING STEAM JACKET.

Indicator Scale, 16 lbs. per inch.

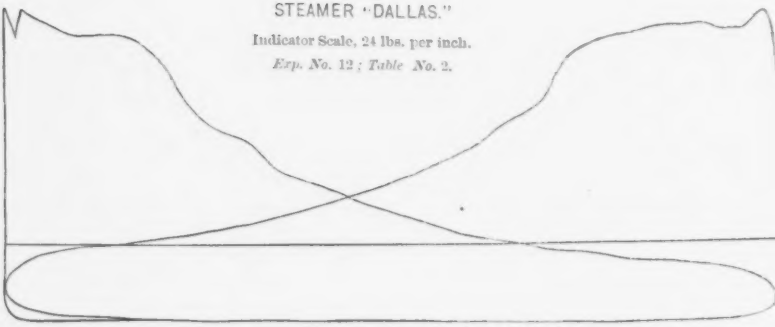


H.

STEAMER "DALLAS."

Indicator Scale, 24 lbs. per inch.

Exp. No. 12; Table No. 2.



fer to the number of the experiment in Table No. 1.

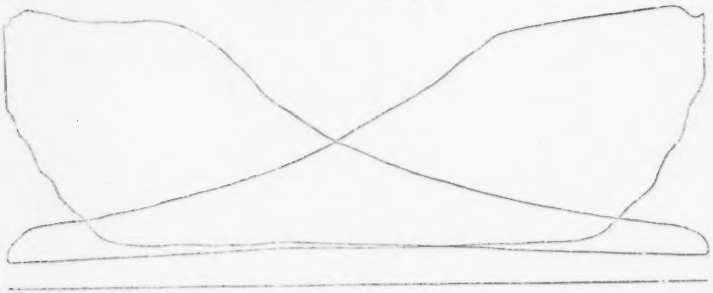
F 1.

STEAMER "RUSH."

HIGH PRESSURE CYLINDER.

Indicator Scale, 40 lbs. per inch.

Exp. No. 1; Table No. 2.



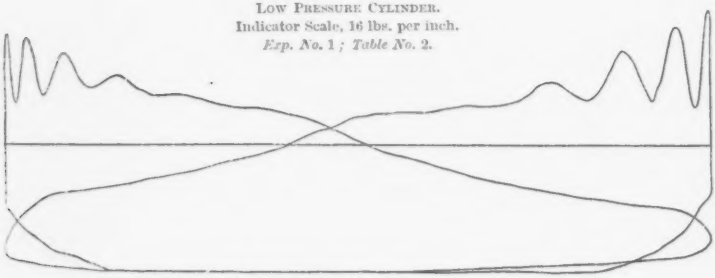
F 2.

STEAMER "RUSH."

LOW PRESSURE CYLINDER.

Indicator Scale, 16 lbs. per inch.

Exp. No. 1; Table No. 2.

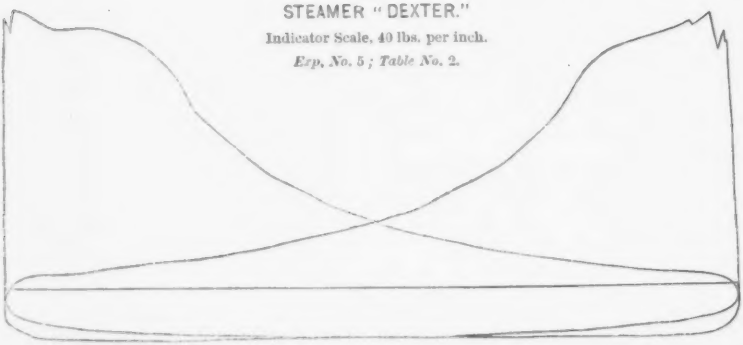


G.

STEAMER "DEXTER."

Indicator Scale, 40 lbs. per inch.

Exp. No. 5; Table No. 2.



slight percentage of moisture. The diagrams show, however, that liquefaction rapidly took place when the walls of the cylinder were alternately cooled and reheated as described above, (3 C). When using the single cylinder of the Bache without steam-jacket (see diagram *B*), the expansion curve for bottom of cylinder fell below the Mariotte curve at first, but afterwards rose above it as re-evaporation took place. In the top of the cylinder, however, some water collected, and its re-evaporation during the expansion portion of the stroke caused the expansion curve to rise above the Mariotte curve at all times. At the highest grade of expansion, with and without use of jacket, a loop was found on the diagram from the upper end of the cylinder in two instances (as shown in diagrams *C*), which disappeared upon opening the cylinder relief valve.

These diagrams show that horizontal engines should be more economical than vertical ones, for the reason that when properly constructed, the cylinder can be kept drained at all times.

When using the single cylinder of the Bache, with jacket (see diagrams *A*), the moisture in the steam due to reheating the surfaces was re-evaporated throughout the stroke, causing the expansion curve to rise above the Mariotte curve in most cases. As before, the difference is greater for the top of the cylinder and very large for the shortest suppressions.

With the compound engine of the Bache, the expansion curve for the smaller cylinder which was not steam-jacketed (see high pressure diagrams *D*), also rises materially above the Mariotte curve. The cut-off valve faces were in excellent condition, making leaks improbable, so the difference is doubtless due also to re-evaporation of moisture. The exhaust from the smaller cylinder of compound engine is not so effective in carrying moisture out of the cylinder as in ordinary engines with less back pressure. The slight superheating of the steam on the Rush, together with the steam-jacket, was sufficient to prevent great variation of the two curves (see high pressure diagram *F*).

When the engine of the Bache was operated as a compound engine, without the jacket on the larger cylinder, there was a material reduction of pressure in that cylinder throughout the stroke, as compared with that during the experiments with steam-jackets in use, which may be observed by comparing low pressure diagrams *D* and *E*, and the mean pressures shown in line 25 of Table No. 1.

Superheating.—In conclusion it may be observed that the diagrams taken at the higher grades of expansion on the Bache show such positive

evidence of the presence of water in the cylinder (due doubtless, as has been stated, to internal condensation), that it may be claimed that by superheating the steam, the ratio of expansion could be economically increased beyond that shown by the Bache experiments. In discussing this question, however (subject 6), the conclusions were founded on the experiments both with the Bache, where there was probably no superheating, and with the cutters where the conditions were favorable for the thorough drying of the steam. From both series, and other experiments available, it is concluded that superheating will undoubtedly reduce the final cost, but it does not appear that it will change in a material degree the most economical point of cut-off for a given steam pressure. Superheating should, in fact, show beneficially in the same way as the steam-jacket and in a measure reduce the advantages shown by the latter.

The variations above referred to between the actual and Mariotte expansion curves, which show, doubtless, the presence of extreme internal condensation, are frequently observed in marine engines using high grades of expansion, and in such cases we may naturally infer, from the whole evidence presented, that there is also a loss of result compared with that obtainable with less expansion. In the experiments with the single cylinder of the Bache, it may be noticed, incidentally, that the expansion curves on diagrams from the bottom of the cylinder correspond closely with the Mariotte curves (particularly with steam-jacket in use), when the engine is working at its most economical grade of expansion, and we have observed a similar correspondence in diagrams from both ends of the cylinder of horizontal engines, where provision was made to drain the water from the cylinder at each stroke.

The Mariotte curve has been used for comparison simply for convenience, without discussing the question whether it is the true expansion curve or not.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE. — This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CV.

ON THE PRINCIPLES OF THE CONSTRUCTION OF, AND THE CALCULATION OF THE STRAINS IN, REVOLVING DRAWBRIDGES* HAVING TWO SPANS OR OPENINGS, AND BUILT AS CONTINUOUS GIRDERS, MORE ESPECIALLY AS CONTINUOUS *PANEL* GIRDERS.

A Paper by CLEMENS HERSCHEL, C.E., Member of the Society.

PRESENTED FOR THE NORMAN MEDAL, SEPT. 2, 1874.

First principles, then rules.

In order that the human intellect may approach the perfect answer in any physical investigation, there is necessary the combined application, each in its proper proportion, of three instruments: mathematics, experiment, and a well-balanced, trained, common-sense judgment.

INTRODUCTORY. — The choice of subject above presented probably needs no apology, before an audience of American engineers. After having neglected the construction of *continuous* fixed spans, with probably good reason in the majority of cases, as we shall see, but on the other hand carried the construction of certain kinds of single fixed spans (link-bridges) to a degree of perfection and size hitherto unknown, American engineers have suddenly been brought face to face with the problem of continuous spans, by the abandonment of the old form of drawbridge, whose ends are suspended from a central tower by chains or rods, and the adoption of the continuous form of truss for such structures. The number of drawbridges of this latter class is steadily increasing, and some of them have already attained a magnitude of span never before attempted; yet it is a painful fact that, in the

* The word "drawbridge" will, in this paper, be used only in the sense of one of that species of drawbridges that are made to revolve around a vertical axis situated between their ends.

calculation of the strains in continuous drawbridges, unsupported opinion and "good enough" approximations have often had a controlling voice; whereas such guides are evidently out of place in engineering or in any other physical science. And it is, perhaps, only a natural consequence of this that there are many unpleasant rumors (facts are difficult to arrive at in such cases) as to the unsatisfactory performances, wear, and life of many of the great drawbridges of the country, both of the old and new styles.

The subject is therefore submitted as one of practical and ample pecuniary interest to the profession and to their employers.

PART I.

§ 1. History of the Theory of Continuous Girders. — The calculation of continuous girders commences with Navier, about 1830, who, it is believed, first propounded the theory of the "elastic line;" that is, the curve or form which the neutral axis of a body, following the laws of elasticity, would assume upon being supported by any number of fixed supports, which were first taken as being all on the same level, and then acted upon by a series of vertical forces or loads. Navier showed how, from the equation of this curve, could be deduced the reactions at each support, the value of the moments over the supports and generally along the line, &c. Still, his method led to somewhat laborious calculations, and was therefore but little used, until in December, 1857, a French engineer, Clapeyron, published, in the "Comptes Rendus," an improvement on the method of Navier, which consisted in finding primarily the moments over each support, and then deducing, from the moments so found, the reactions sought for. Clapeyron's equations required that: 1. The load be uniformly distributed on each span. (The several spans may, however, have different loads.) 2. The supports must all be situated on one and the same level. 3. The moment of inertia of the cross-section of the girder, referred to the neutral axis, must be constant. Other assumptions made by Clapeyron, but which are common to all the theories of the elastic line, before and since his day, will be spoken of further on.

The publication of Clapeyron's article seems to have given a great impetus, in France and Germany at least, to the investigation of the properties of continuous girders, and speedily led to the extension of the theory of the elastic line to the cases where the supports are not on a level,* to investiga-

* First noticed by C. Köpke in "Zeitschrift des Archt. & Ing Vereins zu Hannover," 1856.

tions as to the effect produced by such a change of level in the supports, &c.; and much has since been written in French, German, and English * technical journals, and other publications, with some books partly or wholly on this subject. [See the list given in the Appendix.]

It is characteristic, however, of all the books and articles that appeared on this subject, between 1860 and 1873, that they severally treat of special cases only, at least what in the light of the most recent work † appear as only special cases. Thus Mohr develops the properties of girders, continuous over two or three spans of any relative length, for *uniformly distributed* loads, with the supports on a level, or raised or lowered certain known quantities, and for girders of varying section. Laissle and Schuebler do nearly the same; while Quensel gives the equations for *concentrated* loads, in girders continuous over two spans, when both spans are equal.

It was the first intention of the writer to add to these cases, by developing the equations for concentrated loads, on a girder, continuous over two spans, when the spans are *unequal* and the supports either in or out of level; which then seemed the only set of equations needed to meet the general case here treated of, — a continuous girder drawbridge, loaded and supported in any manner.‡ All such partial investigations have been set at rest, however, by the recent work of Weyrauch, previously referred to, which, in the language of the preface, presents a complete "theory of straight girders, continuous over any number of openings from 1 to ∞, and for any kind of concentrated or regularly or irregularly distributed loads." It even treats the single span girder as but a special case of the general subject above presented. Following a work as exhaustive as this, there remains but little to be done in the way of generalization; and it has been freely used in deducing the equations needed to calculate the reactions at the several points of support of a drawbridge.

The general equations thus taken from Dr. Weyrauch's book, the special equations then derived from them, and their mode of derivation, will be found in Part II.

§ 2. The Practice and Theory of the Elastic Line. — It is time now to speak of the value of all these investigations, of the assumptions upon which they rest, and in how far we can judge or *know* that the results obtained represent the exact truth.

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* The writer cannot state as to works in other than these three languages.

† The work by Dr Jacob I. Weyrauch cited in the Appendix.

‡ However, this would not have met the whole case, as will presently appear; for usually the drawbridge, owing to its being supported at *two* points at the centre, becomes a girder continuous over four supports, or of three spans instead of two.

The theory of the "elastic line," together with the long train of investigations that has grown from it, rest upon certain assumptions to be found in most theories of flexure, and very fully set forth in the work to which reference has just been made. In order that a bridge structure should exactly conform to these assumptions: 1. Every member and every part of every member would have to extend or compress strictly equal distances by the application of equal strains per unit of section. 2. If the girder were laid over on its side, — that is, not be acted on by what ordinarily are its vertical forces, — every part of it would have to be without any strain whatever. 3. When set up, no part of it must in the slightest yield otherwise than according to 1. 4. Should it act as a continuous girder on supports that are on a level, its actual supports must, when the bridge is set up, conform strictly to the profile of the bottom of the bridge when it is in condition 2. This is a point of some importance, as will appear; and it is well perhaps at once to clearly catch the idea of what is meant by "supports on a level" and "supports so and so much out of level," speaking after the manner of continuous girders. In point of *absolute* level, the supports may all be on a different grade and yet react as supports that are on a level, if the girder, when without strain or when it is acted upon by the calculated theoretical reactions, has an under profile fitting to this broken grade line of the supports; and, if mention should be made of such a support being lowered so much out of level, it would mean, lowered so much from this primary grade line, &c. 5. In the theories generally given it is assumed that the moment of inertia of the cross section of the girder referred to the neutral axis is a constant. There are, to be sure, methods for calculating the reactions, moments, and strains in girders whose cross sections, and therefore moments of inertia, vary by steps, or according to known gradual increments or decrements, accurately as well as approximately, but they are both so laborious as to make their use unwarranted, save in rare cases; and even then the uncertainty and approximations inherent to the theory in general are not removed by this additional accuracy of taking into account these variations. It is safe to affirm, namely, that in actual practice none of the conditions 1, 2, 3, and 4 ever strictly obtain. Under 1, different pieces of iron do not extend and compress strictly as the strains upon them; each separate plate, angle, T, or channel will vary more or less in this respect within its own length and each with the other. 2 is a condition which must be different in different forms of construction, link or riveted, or nut and rod, and is probably never absolutely attained in practice. 3 depends on 2 in the first place, and even starting with 2 perfectly attained it is just as

reasonable to suppose that more or less members will yield somewhat, no matter how little, other than by extension or compression of the material they are composed of, as that they will not at all, especially on being first brought into action; and, again, that different forms of construction will act differently in this respect. 4 is a condition of which, ordinarily, really nothing is absolutely known; it is of the more importance, as very small changes of level produce a great increase or diminution of strains. How this condition may be regulated will be shown farther on. 5 has already been discussed.

There would seem to be, therefore, no particular reason why we should give implicit faith to the results of the theories based on that of the elastic line, any more than we do to those derived from theoretical hydraulics; we can, on the contrary, as a matter of judgment, gravely question their accuracy, and, so far as known to the writer, there are yet wanting any experiments on finished structures from which coefficients of correction, such as are indispensable in the science of hydraulics, could be deduced to apply to the science of bridge-building, more particularly to that of continuous girders.

Another difficulty lies in the form of the equations presented to the engineer for calculating the dimensions of a desired girder. If the supports are not "on a level," or if it is desired to apply the more exact formulæ, which make allowance for the variations in the cross section of the chords, it will be found that these very equations already contain the desired dimensions, algebraically of course. There is then nothing left to do but to apply the method of successive approximations; that is, first to find the dimensions as well as may be, introducing the value of the dimensions sought in the equation used to find them, according to the best judgment of the calculator, then from the structure so designed to start anew. There is no desire here to overrate the deficiencies of the system as it stands developed to-day, nor to conceal the fact that multitudes of bridges have been built in accordance therewith and are doing good service. The question that remains is: inasmuch as the formulæ do not give exact mathematical results, how near are they to the truth for various kinds of bridges or what are the coefficients of correction? Happily, much uncertainty could be eliminated, and the two last described calculations dispensed with, were the method of *weighing off the actual reactions* of the finished bridge once substituted for that of calculating the proper position or level of the supports, in order that certain reactions might be obtained. To arrive at this method in the development of the proper manner of calculating drawbridges seems very natural, but it

is nevertheless somewhat puzzling that, with the great care and expense applied to the erection of the numerous continuous girders that have been built in France and Germany, this simple method should not have been thought of and practised rather than the uncertain one of calculating minute differences of level of the supports, and then attaining them as nearly as possible.

To illustrate, take a girder of uniform section, uniformly loaded, of two *equal* spans, supported on three supports all on a level.

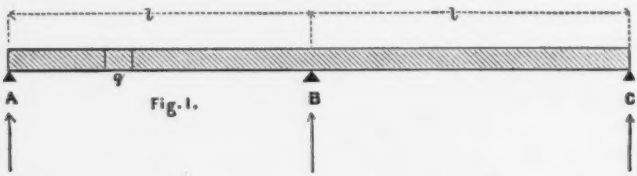

Fig. 1.

A, B, C, are the reactions, q the weight per unit of length, therefore from **49** and **50**,* $A = C = \frac{3}{8}\, ql$, $B = \frac{10}{8}\, ql$, when all the conditions of a continuous girder are fulfilled. But if A and C are each lifted up by a lever or set of levers (the levers ultimately to be kept in position by, say a spring dynamometer) or by any other weighing apparatus more or less approximate,† until these scale beams read $\frac{3}{8}\, ql$, and permanent wedges are then driven under A and C until the spring dynamometers or temporary supports return to indicate 0, we shall be certain that we have left the girder acting as a theoretically perfect continuous girder, under that load at least. Again, if the reactions left under the girder were only those for which it had been calculated, it would be a correctly designed structure, no matter whether these reactions were those theoretically due to a continuous girder on level supports or on supports to a certain degree out of level. For a load q_1 on one span, and q_2 on the other, the theoretical reactions on level supports would be (**46** — **48**) —

$$A = \frac{l}{16}\,(7q_1 - q_2),\ B = \frac{10\,l}{16}\,(q_1 + q_2),\ C = \frac{l}{16}\,(7q_2 - q_1),$$

and further experiment would show whether the same girder, reacting theoretically perfect under the one kind of load, did so under the other kind also, or, if not, what were the actual reactions. From a series of such tests, valuable practical data would speedily be obtained; and in the mean

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* See equations in Part II.

† A good hydraulic press that had previously been experimented with and duly rated would make a convenient and direct weighing machine.

while girders would have to be so supported that they should receive the proper reactions when under the loads which produce the most frequently occurring maximum strains.

§ 3. Some Properties of Continuous Girders. — Before proceeding to speak specifically of only continuous drawbridges, it may be well to become more fully acquainted with some of the properties of continuous girders in general. One of their most remarkable characteristics is the effect they undergo by being supported on supports which are not "on a level." The equations that give the reactions, maximum moments, &c., under these conditions, all contain the moment of inertia of the girder cross section and the ordinates below a level line of the several points of support. Likewise, with given sections of girder, spans, and loads, may be found the ordinates of supports that will produce desired strains and moments. See equations **32–34** containing the term Y, or **59–62** containing Y_1 and Y_2, and others. A large part of most treatises is taken up with the calculation of these special ordinates; that is, ordinates producing special strains and moments, and the result of some of these calculations it will be instructive to look over. It is proposed in this paper to make no use of any formulæ containing the terms Y — that is, the ordinates of the support and the moment of inertia of the girder cross section — for several reasons: 1. to abbreviate calculation; 2. because these ordinates cannot be measured practically with the nicety that the equations demand, — thousandths of a foot in difference of level usually make tons of difference of reaction and many tons' difference of strain; 3. because the introduction of these terms, as has been stated, fits the equations indeed for finding the strains in a designed girder, but not for finding the correct dimensions of a girder about to be designed; 4. because by directing our energies toward a different object — the weighing of reactions — more can be achieved in the investigation of the same subject. In the present state of the science of continuous girders, certainly the value of the deductions about to be given lies rather in the warning they offer as to the effects of accidental or unknown differences of level of the supports, than in the use to be made of them and the formulæ they are derived from, in the designing of new works. The following tables, I. for two spans and II. for three spans, are taken from the work of Laissle and Schuebler, and will explain themselves. The object aimed at is to make the cross section of the chords as nearly uniform as possible; that is, equal in maximum in each span to what it is over the centre support in the case of two spans, and equal in all three spans to what it is over the centre supports in the case of three spans.

TABLE I. (*for* 2 *Spans*).

Length in ft. of each span		23.	32.8	49 2	65.6	98.4	131.2	196.8	328.
Ratio $\frac{p}{q}$		$\frac{1}{6}$	$\frac{1}{7}$	$\frac{1}{5.3}$	$\frac{1}{4.5}$	$\frac{1}{3.6}$	$\frac{1}{3}$	$\frac{1}{2.5}$	$\frac{1}{1.9}$
The supports level.	Max. moment in the 2 spans	0.0927	0.0919	0.0906	0.0897	0.0882	0.0868	0.0851	$0.0819ql^2$
	Max. moment over the centre pier	0.125	0.125	0.125	0.125	0.125	0.125	0.125	$0.125ql^2$
Centre supports depressed to most advantageous position.	Max. moment at centre support and in the two spans	0.1026	0.1019	0.1011	0.1004	0.0994	0.0983	0.0970	$0.0946ql^2$
	Depression in ft. of centre support $= s$	0.0111	0.0164	0.0247	0.0357	0.0563	0.0815	0.1262	0.2468

TABLE II. (*for* 3 *Spans*).

Length of middle span in ft. $= l_2$.	$\frac{l}{l_2} =$		$\frac{1}{2}$	$\frac{3}{4}$	$\frac{4}{5}$	$\frac{5}{6}$	$\frac{7}{8}$	$\frac{9}{10}$	1
	$\frac{p}{q}$ =								
	1	$s =$	$0.00042l_2$	$0.00119l_2$	$0.00148l_2$	$0.00171l_2$	$0.00205l_2$	$0.00182l_2$	$0.00092l_2$
	$\frac{3}{4}$	—	0.00039	0.00100	0.00123	0.00142	0.00169	0.00155	$0.00083l_2$
574	$\frac{2}{3}$	—	0.00037 .2123	0.00093 .5338	0.00114 .6543	0.00132 .7576	0.00153 .8782	0.00146 .8380	$0.00078l_2$ or .4477 ft.
328	$\frac{1}{2}$	—	0.00035 .1148	0.00082 .2690	0.00100 .3280	0.00115 .3772	0.00133 .4362	0.00132 .4329	$0.00072l_2$ or .2379 ft.
230	$\frac{2}{5}$	—	0.00033 .0759	0.00076 .1748	0.00092 .2116	0.00106 .2438	0.00123 .2829	0.00123 .2829	$0.00068l_2$ or .1564 ft.
148	$\frac{1}{3}$	—	0.00032 .0473	0.00072 .1065	0.00087 .1287	0.00100 .1480	0.00115 .1702	0.00119 .1761	$0.00064l_2$ or .0947 ft.
82	$\frac{1}{4}$	—	0.00031 .0254	0.00066 .0541	0.00080 .0656	0.00092 .0754	0.00106 .0869	0.00111 .0910	$0.00063l_2$ or .0517 ft.
60	$\frac{1}{5}$	—	0.00031 .0186	0.00064 .0384	0.00078 .0468	0.00089 .0534	0.00102 .0612	0.00108 .0648	$0.00063l_2$ or .0378 ft.
40	$\frac{1}{7}$	—	0.00030 .0120	0.00061 .0244	0.00073 .0292	0.00082 .0328	0.00099 .0396	0.00104 .0416	$0.00059l_2$ or .0236 ft.
25	$\frac{1}{9}$	—	0.00030 .0075	0.00059 .0147	0.00071 .0177	0.00079 .0197	0.00093 .0242	0.00101 .0252	$0.00053l_2$ or .0132 ft.

In these tables:

p = own weight per ft., q = live load per ft.,

l = span in ft., both spans equal in Table I., two outside ones equal in Table II.

l_2 = span in ft. of middle span in Table II.

The height of the truss is supposed to be $\frac{1}{10}$ of l and $\frac{1}{10}\frac{2\,l+l_2}{3}$ for Tables I. and II. respectively, with a strain of about 8500 lbs. per $\square''$ in the chords.

Mr. Mohr, in his article, 1860, finds for a strain of about 8,400 lbs. per $\square$ inch, two spans, height $\frac{1}{10}$ of span, modulus of elasticity = about 29 million lbs. per $\square$ inch: $1+z=1+51\,\frac{y}{h}$, where z is percentage of strain in the chords at centre, due to lowering of centre support, y is amount of such lowering, and h is the height of girder, both expressed in same unit of length. We find, therefore, that for a lowering of the centre support equal to $\frac{1}{51}$ part of the height of the girder, the strain in the chord at the centre is already doubled.

Similarly, for three equal continuous spans, the same admirable writer finds $1+\frac{10\,z}{7-r}=1+69\,\frac{y}{h}$, where $r=\frac{\text{own weight}}{\text{live load}}$, each per unit of length. Such a girder would therefore require that the central supports should be only $\frac{1}{69}$ of the height of the girder lower than the outside ones, or that the bridge, when laid on its side and every member without strain, should have a crown at the points where it will rest on the centre supports, above a straight line connecting its two end-supported points of $\frac{1}{69}$ of the height of the truss, in order that it receive a strain in the chords over the centre supports of $2\times 8{,}400=16{,}800$ lbs. per $\square$ inch. Only half the described deviation of level in the piers, taken in conjunction with half the described crown of the bridge, will produce a like effect.

*For small bridges, especially, the measurements of level required to properly mount a continuous girder are practically unattainable,** and the advantage to be derived from lowering the centre supports is completely swallowed up in the disadvantages resulting from accidental or unknown changes of level. The most advantageous level for the central supports of a three-span continuous girder, as compared with three single spans, will result in a saving of some 18 — 39% of chord section; but an equal disadvantage

* It may not be out of place to call attention to the fact that this difficulty decreases for large spans, and again to note that it can always be entirely avoided by weighing the reactions instead of measuring the relative level of the supports.

arises from a variation of level in the points of support of $\frac{1}{21000}$ to $\frac{20}{21000}$, that is, for small bridges of $\frac{1}{1050}$ and for larger ones of $\frac{1}{105}$ of their own height.

An endeavor to calculate continuous girders, without having recourse to the measurement of the distances the several supports are "out of level," would therefore seem to be well warranted. The same remark is less true as regards omitting the consideration of variations in the moments of inertia of the chord sections. The time for this additional approximation will come, when we shall have obtained coefficients of correction from experiments with many kinds and qualities of finished bridges. It will be to these last, most nearly correct formulæ, that the coefficients will have to be applied. At present, the methods to which this paper has been limited can gain but little additional *accuracy*, by having regard to variations of the moments of inertia, so long as they contain the relatively equal *inaccuracy* of being without practical coefficients of correction. In the mean while it is gratifying to find that calculating without regard to moments of inertia of the chord sections is of the less importance the more nearly the actual sections are made exactly proportional to the strains in them. For the mathematical case, where the above proportion is exactly obtained, Mr. Mohr finds for two equal continuous spans, uniformly loaded: —

$M_2 = 0.146\ ql^2$ = moment over the centre pier,

$A = 0.354\ ql$ = reactions at ends, and

$M' = 0.06215\ ql^2$ = maximum moment in each span, when the change of moment of inertia is taken into account; and, as is well known,

$M_2 = 0.125\ ql^2$,

$A = 0.375\ ql$,

and $M' = 0.0703\ ql^2$, neglecting the same.

Similarly, for three equal continuous spans, uniformly loaded, the moment over the central piers, the reactions at the ends, and the maximum moments in the first and third and middle spans are found for the two cases of varying and of uniform moments of inertia: —

$$\left.\begin{aligned} M_2 &= 0.1067\ ql^2, \\ A &= 0.3933\ ql, \\ M' &= 0.0773\ ql^2, \\ M'' &= 0.0183\ ql^2, \end{aligned}\right\} \text{ and } \left\{\begin{aligned} M_2 &= 0.100\ ql^2, \\ A &= 0.400\ ql, \\ M' &= 0.080\ ql^2, \\ M'' &= 0.025\ ql^2.* \end{aligned}\right.$$

§ 4. Loads and Reactions of Continuous Drawbridges. — Pivot drawbridges, when constructed as link-bridges, usually have *two*

* See also the article by J. W. Schwedler, 1862, p. 277, cited in Appendix.

centre posts at equal distances each side of the pivot or centre. Indeed, whatever the construction of the girder, two supports at the centre will be the rule, and one only, exactly at the pivot, the exception. In the latter event, the formulæ for two equal and those for two unequal spans, for the several kinds of loading, will give the reactions for any case that may arise. It is proposed to discuss in full only the more difficult case of *two* centre supports, in the course of which the other and less usual case cannot fail to be likewise sufficiently elucidated, the more so as the two have many points in common. Take, first, the supposition of symmetrical loading only; that is, loading such that no tendency to pivot at the centre exists, and let us see the effect on the truss, of the several possible levels of the end supports as referred to that of the two centre ones. The former couple can always, and the latter two can for all cases of symmetrical loading, be considered as though on a level.

Case I. The bridge open and loaded with snow. In Southern climates, and in those cases of railroad bridges that have no flooring, the snow load may of course be omitted. In this latitude (40° and above) and for highway bridges it cannot be overlooked. The end supports are wanting, the bridge being open, and we have a girder resting on two supports only; and the reactions at each must necessarily be equal to half its own weight, plus the weight of the snow on half its length.

Case II. The bridge shut and loaded with snow plus the maximum live load. It will be evident that the reactions at the four points of support depend now very much on their relative levels. If the end supports are wanting, — one extreme case, — the girder would be in the same condition as when open, as regards support, but subjected to the maximum loading; if the end supports were lifted so high as to lift the ends of the girder higher than the two central supports, by an amount exceeding or just equal to the deflection of the girder, acting as a single span-bridge between the two end supports when subjected to the moving load, — the other extreme case, — it would of course act as such a single span girder alone, and undergo a strain at the centre equal to what it would in *Case I.*, under an equal load and were there only *one* central support, but with a general reversal of the *directions* of all strains. Both these cases are, however, practically out of question, and exist only as logical possibilities; * but between them lie an infinity of

* The first is too outrageous a case of wrong for even an unconscientious draw-tender to permit; the other could be prevented by not furnishing said draw-tender with any apparatus to lift the bridge off its centre supports. In the case of railroad drawbridges whose ends are intended to be lifted, the passage of trains before such lifting has taken place may effectually

supposable cases. Having eliminated from the discussion the consideration of measured differences of level of the supports, and substituted therefor that of the actual weights of the reactions, we next eliminate this latter by considering only the several cases when the end reactions are just equal to 0.* It will readily be seen, therefore, that for the case in hand, a bridge loaded with snow and a live load, the reactions at the ends may be made $= 0$: when the bridge is empty, when loaded with snow, or when loaded at its maximum, though this last brings us again to one of our suppositions that may practically be neglected. *Or any given number of tons or lbs. per unit of length or panel may be considered as acting on the girder continuously, and the balance will be supported at the central points, which is equivalent to saying that, when loaded with this balance, the skeleton outline of the girder will endure no reactions at the ends.* It is evident that, by thus dividing up the loading into two parts, one acting on the girder continuously, the other supported at the two central points alone, the total reactions at the four several points can in all cases be obtained.

Another way to explain this process of finding the total reactions by first finding the reactions due to certain elements of the loading is this; and the remarks above, relative to what constitute "level supports," and what "supports out of level," speaking after the manner of the elastic line theory, must be borne in mind in this connection. Take, first, the girder loaded with any part of the total load q, and the end supports then brought just into contact with the ends of the girder. For all *future* loads, the reactions will be the same as those due a *straight* girder on level supports; in point of fact, it is a *curved* girder on supports just fitting it, which amounts to the same thing. Sometimes the strains in each *member* have been calculated in parts, due to different loads, and these then summed up; the plan submitted of summing up only the *reactions* in parts, and then getting the strains of members at one operation, is generally to be preferred.

A practical choice, for the present, will be to suppose the end reactions equal to 0 or end supports barely in contact, when the bridge is empty. It will shut a little hard in that case, when loaded with snow; but, as the draw is opened least about the time that heavy snow-storms occur, this will only be taking advantage of a convenient provision of nature, and not working

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be prevented by connecting the signal that the bridge is ready for travel with the bolt that bolts the draw ends into the fixed bridge, and then arranging this bolt so that it cannot be shut until the draw ends *are* lifted. Still, the ingenuity with which such labor-involving contrivances are sometimes circumvented is fully equal to that which produced them.

* Attention is here called to the three widely different cases of having a reaction $= 0$, a *plus* or a *minus* quantity.

against one. Certain drawbridges, as will appear, have the ends raised before being subjected to travel; but even for such it is proper, unless the draw-tenders are under rules no whit less strong than military rules, to allow for the case where travel goes upon the bridge *before* the draw-tender has lifted or sufficiently lifted the ends of the draw; in other words, always to take the reactions at the ends for the empty bridge just equal to 0 in calculating *Case II.* We shall have, then, the reactions at the ends due to a load equal to the live load of the bridge + snow load acting on a girder continuous over four supports, or to be derived from equations **51–76**, for the several kinds of loading and proportions of length of spans one to the other. To the centre reactions derived from these equations, add the weight of each half girder, to get the total reactions at the two central supports.

These two, *I.* and *II.*, constitute all the cases of symmetrical loading, and we next take up the more numerous cases of unsymmetrical loading. This leads at once to a discussion of the manner of support of the two central supports (of the nature of the turtle, that is under the elephant, that supports the world). We shall distinguish two sub-cases: (*a*) when the two central supports remain firm, the same as though they rested upon a solid, inelastic, stone pier, or as though the draw took bearing upon the circular girder, which in turn rests upon the "live ring"; and (*b*) where the live ring is not considered as offering support, and the whole weight is thrown, by suitable framing, upon the centre pivot.* It will be readily seen, also, that any deflection that may occur in this "framing" may be neglected: we have but to lower the end supports an amount equal to this deflection, to render it entirely nugatory.

For equal outside spans, which alone it will be necessary to discuss, (unequal spans are treated just like equal ones, only that the short span is loaded with concentrated loads sufficient to balance its deficiency of own weight), there will arise twelve different cases of unsymmetrical loading, belonging under (*a*) and (*b*) to three different styles of drawbridge, and according as the snow load is considered or not. We find, namely, that for unsymmetrical loading, load on Am (m is point at centre of bridge) whether sub (*a*) or (*b*), we very often get the reaction at D first $= 0$, then a minus quantity. That is, in the latter event, in order that the elastic line shall remain in contact with the end support opposite to the loaded side, the light end must be weighted or force of some sort must be applied to keep it in

* In some drawbridges the centre pivot is used only as an axis of rotation, never as a support; in such, of course, the cases (*b*) cannot arise.

position. In the absence of such provision, it will rise or "kick up," especially if the loaded end is not very firmly supported. Now it requires but a little thought to see that so and so many tons, suddenly rising ½ inch, 1 inch, or 1½ inches, &c., many times a day, and as suddenly falling again, and with nothing else to spend their work upon, are only so many foot-pounds assiduously laboring to destroy the bridge or the piers. Such a structure partakes too much of the nature of a tilt-hammer to be entirely satisfactory as a permanent drawbridge. Upon taking up therefore the question as to how to prevent this tipping, we find two methods in practice, the first long in use, the second apparently for the first time used by Mr. Charles Macdonald of New York. They may be likened to the two methods at command for preventing a horse or an ox or a man from kicking. One way would be to fasten his heels to the floor, the other to lift them up so high that any attempt to kick would indeed diminish the strain upon the chords that held said heels up, but would never result in their going any higher. In the same way may be managed this restive end of unsymmetrically loaded drawbridges: it may either be lifted up so high that the reaction D shall under no case of loading become either equal to 0 or a minus quantity, or it may be fastened down, this last being Mr. Macdonald's way.

As previously remarked, and as often found, the ends may also be left free; and we have therefore the above-stated twelve cases (two of each kind, according as we consider the snow load or not) of unsymmetrical loading as follows:—

Case III. (*a*) ends free, on firm central supports.
,, *III.* (*b*) ,, ,, ,, pivoted ,, ,,
,, *IV.* (*a*) ,, latched, ,, firm ,, ,,
,, *IV.* (*b*) ,, ,, ,, pivoted ,, ,,
,, *V.* (*a*) ,, lifted, ,, firm ,, ,,
,, *V.* (*b*) ,, ,, ,, pivoted ,, ,,

Case III. (*a*). Let us again so place the end supports that when the bridge is empty they shall be just in contact with the bridge, no more and no less; that is, their reactions shall equal 0. Imagine now a girder so supported, no part latched down, to be loaded on length Am (m being point at centre) with dead + snow + live, and on length mD with dead + snow. This will generally (it may be proven in each case) give the maximum *absolute* strains in the chords at least. It would be *relatively* more severe for the two main spans of the girder, to suppose one side loaded with dead + snow + live, and the other with dead only; but there is a little of the absurd in thus

supposing a bridge only half covered with snow, and at the same time it does not furnish absolute maxima. We take therefore, in all the unsymmetrical cases of loading, for maximum strains, dead + snow + live on Am, and dead + snow on mD. The snow load has, however, an important effect in forming the nature of the end reactions and of some of the webbing in some cases; and it is necessary carefully to consider the bridge with and without snow upon it, as it may easily happen that a bridge that is firm under snow + live on one side, and snow on the other, becomes tipping and unsatisfactory with the snow load removed. To determine the nature or positions of the end supports and in some cases certain members of the webbing, we therefore calculate all the unsymmetrical cases for dead + live on one half and dead only on the other, as well as for snow added to each. In the case of panel bridges having only one panel over the centre, half the centre panel load can be supposed to be applied at the inner extremities each of spans AB and CD: if there should be more panels, the formulæ will easily admit of being extended to cover those cases also; and, for plate girders and uniformly distributed loads, the load on centre span can likewise, if desired, be taken into account, so as to make in all the unsymmetrical cases the load on just half the length of the girder.

To resume, then, the special case under consideration, we have a girder on four supports, unequally loaded (Fig. 2), and B and C higher than A and D by an amount just equal to the deflection of the bridge under its own weight when resting on B and C alone, no matter what the exact depth of such deflection be in decimals of a foot or inches.

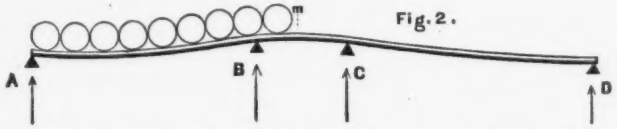


Fig. 2.

First, take the action of the bridge under its dead load only. Reactions at B and C are each equal to half of dead on whole girder. Now place end supports in contact. For all *future* loadings the bridge is equivalent to a straight girder on level piers. Add snow + live on Am and snow only on mD. Find reactions for this unsymmetrical load from **63 - 66** (or **69 - 72**), four supports on a level, and to B and C previously found add (algebraically) the new B and C for the final B and C. A and D are found at once and directly. Now see whether C and D, one or both, are plus or minus. We can have both plus (α),* C minus and D plus (β), (D minus

* Equivalent, if D is large enough, to *Case V.* (α).

and C plus is impossible), and both minus (γ). If (α), the above calculation was correct; if (β), it is a case of two unequal spans AB and BD. Start anew (Fig. 3) this time with the whole of the dead load of the whole girder on B.

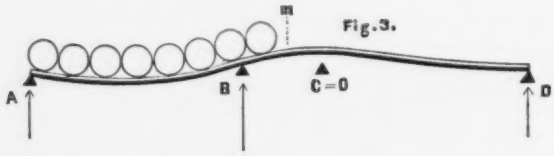


Fig. 3.

But, if put in that position, it would need still another force or better couple to keep it there, since BD is longer than AB, and the dead load is supposed to be symmetrical about m. Such a balancing couple could be applied at A downwards like a load, and at B upwards like a reaction, — that is, *diminishing* any A to be found hereafter, and as much *increasing* B; or at D upwards and B downwards, — that is, *increasing* any D to be found hereafter, and as much *diminishing* B; or finally at both A and D, with the same respective characteristics. Such force is without doubt applied at both A and D; and, to find it, we must first see how much of the unsymmetrical load on Am, when the supports are as in *III.* (α) (α), will just make $C=0$, which can be done from **63 – 66**. For this much of the unsymmetrical load and dead + snow load, the reactions are as in *III.* (α) (α); for the balance of the unsymmetrical load, they are according to **32 – 34** (or **43 – 45**), for two unequal spans AB and BD. This is evidently a rare case, and moreover moves between very narrow limits of possibility. It would require a proportionally very large live load, and peculiar ratios of loads and lengths, to produce such a case: it is given, however, for the sake of completeness, and well illustrates the general adaptability of the method of separating the effect of combinations of loads into the effects of certain elements of the same.

Case III. (α) (γ). If both C and D are minus (Fig. 4), it is a case of one span and an overhanging arm BD. We now have again the whole of

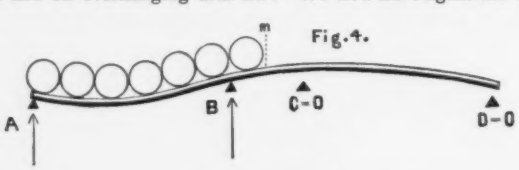


Fig. 4.

the dead load upon B, and the reactions required to balance it about B can be applied at A and B only. These will tend to diminish any future A, and

will equally increase B. The snow will now act precisely as does the dead, and the live will be distributed between A and B according to the simple and well-known laws of the lever. Necessarily A and B are the only reactions. This, again, is a rare case, and requires still greater live loads in proportion to the dead load and snow. Equations to meet *III.* (a) (β and γ) could be evolved by introducing the conditions C, or C and D, $= 0$, and the known values of moments that that implies, in the equations of the elastic line. See book of Weyrauch, often cited.

Case III. (b). — Fig. 5. The end supports are so placed, that for the empty bridge, they shall again be just in contact: dead + snow + live on Am; dead + snow on mD. For the dead load, B and C again each equal half dead load of whole girder. Now find A, B, C, and D from **89 – 91** for live on Am, nothing on mD; also, A, B, C, and D for snow over whole

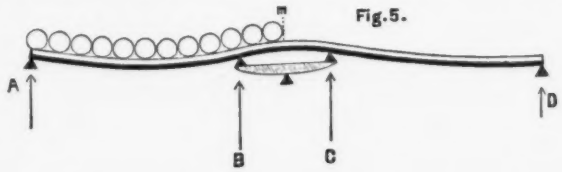


Fig. 5.

bridge from **63 – 66** (four supports on a level).* C cannot now be a minus quantity: it must equal B, in order that there shall be equilibrium, and there remains only to see whether D is or not. If plus (a), the reactions found are correct; if minus (β) (Fig. 6), take first again B and C each equal half dead load of girder. Then, snow over whole girder from **63 – 66** (four level supports), then find (from **89 – 91**) what part of live will just

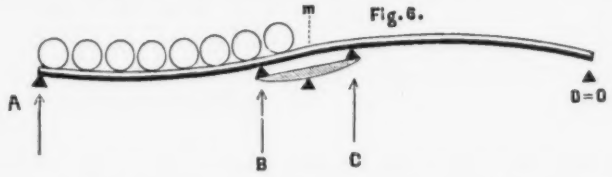


Fig. 6.

make D equal to 0. The reactions for that part are found from **89 – 91**; those for the balance of live from **93 – 94**. Or, a shorter way, take first dead on B and C. If now, in such a pivoted girder, forces ΣP on AB

* The two operations just described with different formulæ could be done in one operation and by only one set of formulæ by introducing into **89 — 91** the terms for loads on mD as well as on Am only.

cause D to become $=0$, that part of the symmetrically distributed load which is on CD will just balance the part on AB, about the pivot under B and C, which will leave the ΣP acting only according to **93–94** on A, B, and C, and the symmetrical loads must all rest in equal parts on B and C.

Case IV. (*a*). — Fig. 7. The end supports are so placed that for the empty bridge they are just in contact. Centre supports firm. Ends latched down. Loading as before. We have, first, half of dead on B and C respectively. From **63–66** find A, B, C, and D for snow + live on Am, snow

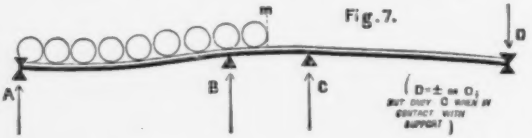


Fig. 7.

only on mD, and add middle terms to B and C already found. It matters not now whether the resultant D be plus, minus, or 0: in either event there is a body at hand to produce the needed reaction. If C is plus (*a*), the reactions are correct; if C is minus (*β*), Fig. 8, it is again a case of two unequal spans, AB and BD. Start again with B and C each equal to

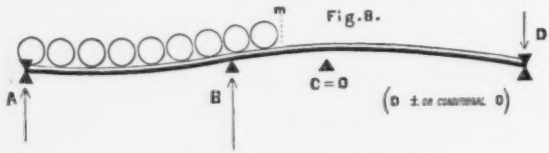


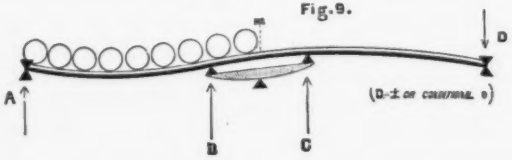
Fig. 8.

half of dead on whole girder. Find and sum up A, B, C, and D for snow on whole girder, from **63–66**; now find what part of live on Am will make $C=0$ from **63–66**; that part will produce reactions, also to be found from **63–66**, the balance will act on A and B only, according to equations **32–34**, for loads on two unequal spans. This also may be considered as a rare case in actual practice. The minus values of D will show the necessary strength of the latching apparatus.*

* The latching apparatus must be constructed to resist, also, a strain that will arise, due to a difference of temperature in the two chords, tending to raise the bridge ends, i.e. top chord warmer than bottom chord. See C. Shaler Smith's paper cited below. The effect of general changes of temperature on the strains and shapes of trusses is very lucidly treated in Ritter's book, cited in the appendix; and a study of the principles of the same would, no doubt, soon lead to the method to be followed in computing the strains and changes produced by a difference of temperature between the two chords of a bridge.

Case IV. (*b*). — Fig. 9. The end supports as before. Centre supports pivoted. Ends latched down. Loads as before. We have, first, half of dead on B and C respectively; then, for snow on girder, A, B, C, and D,

Fig. 9.



to be found from **63–66**; also live on Am, with nothing on mD, A, B, C, and D, to be found from **89–91**; sum up, and we have the final A, B, C, and D.

The relative merits of *IV.* and *V.* it is proposed to discuss further on.

Case V. (*a*). End supports on such a level, relatively to B and C, that they are more than high enough to prevent either A or D from ever being $= 0$. Centre supports firm. Loading as hitherto. Now as to the amount the ends should be lifted. The only object of lifting them can be either to prevent what we have called the "kicking" of the light end or to regulate the strains over the centre. We have to deal with the first only at this time. Find, therefore, with what loading of the whole girder the light end reaction is just equal to 0, when the heavy side is loaded with its maximum moving or live load, and then make the end reactions $= 0$ at a loading *greater* than this, so as to attain a certain factor of safety against such an occurrence. This subject is treated of, for both the case of the pivoted draw and of two unequal spans, in §§ 13 and 14 of Part II. To illustrate, in the numerical example *V.* (*b*), if the ends are raised (see § 13), so that 2.24 tons* per panel react on A, B, C, and D, as a girder continuous over four supports, with B and C pivoted, it will make D equal to 0, when Am is in addition loaded with 12.6 tons per panel. In such a case we would take, say 3 tons per panel, as the uniform load, of which the end supports must carry their due share, in order that the live load on Am shall never cause D to equal 0. And it may be noted here that railroad bridges, subject to l ving trains move over them at a high rate of speed, would need a larger f or of safety against this occurrence than roadway bridges.

In our calculations we therefore take the end reactions equal to 0, when the skeleton lines of the girder are loaded with the own weight, minus the

* Wherever the word "ton" is used in this paper, it means 2,000 lbs.

weight selected as above, which we will call s (for stability), and proceed.* Just as s must now always make D plus, it can and should be required to always keep C plus. The case is therefore analogous to *III.* (*a*) (*a*), only substituting dead $-s$ for dead, and snow $+$ live $+s$ for snow $+$ live, and similarly.

Case V. (*b*). End supports lifted a certain amount higher than for *I.–IV.* Centre supports pivoted. Loading as hitherto. s is found from § 12, Part II., which reduces this case to *III.* (*b*) (*a*). Or find A, B, C, and D from **63–66** (two central supports pivoted), for snow $+$ live $+s$ on Am and snow $+s$ on mD. D will surely be plus, if s has been taken right, and after adding half (dead $-s$) on whole girder, to B and C each, we have the final A, B, C, and D.†

§ 5. Practical Considerations in selecting the kind of Supports for Drawbridges. — The subject, as left in the last section, is perhaps seemingly complex, but seemingly only. We had in all: *I.*, *II.*, *III.* (*a*) (*a*, β, & γ), III. (*b*) (*a* & β), IV. (*a*) (*a* & β), IV. (*b*), V. (*a*) (*a* & β), and V. (*b*), — that is, fourteen cases, or, adding those without snow, twelve more, in all twenty-six cases; but this is only because of the generality of the treatment of the subject, which adapts it to *all* cases of double centre post drawbridges that may occur. All those marked *III.*, in all ten cases, need never, and it is advised should not, occur, or at least reduce to *V.*; that is, have D equal plus. For any particular loading and form of truss, only one of a, β, or γ can ever prevail, and the choice of end support determines whether to use *IV.* or *V.* For any one drawbridge, therefore, we shall have, generally, only four cases of loading to consider; and if it is desired to calculate for *both*, firm centre supports, and for the two central supports pivoted, this will add two more cases, making in all six.

To calculate for partial loadings of any one span will not be necessary for the chords. This can be proved mathematically: numerical proofs can be derived from the equations given, for the construction of the formulæ render the introduction of a single load at any point or distributed loads of various

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* s may be taken 0, or even a minus quantity, as far as tipping is concerned, if the dead load is so great, in comparison to the live, and the length of spans are so proportioned, that the dead alone prevents D from being equal to 0. This weight s should also be such that the raising of the ends due to it will make D plus, when the draw chords are unequally affected by temperature, and the draw ends tend to rise on that account, that is, where the bottom chord is warmer than top chord. See C. Shaler Smith's paper cited below.

† It is proper to remark that the snow load, or, more exactly, the fact that the snow load reactions are not equal to 0, when the bridge is shut, have introduced a triple element into all the unsymmetrical loadings. By leaving out the snow load, as in the second series of cases of unsymmetrical loading, or by making the reactions equal 0 with the snow load on, the several deductions are simplified.

values per unit over any length in any position an easy matter. Instead of ΣP substitute for distributed loads $\int qdx$ between the limits that the load q is distributed. In case of the webbing, it may occasionally be advisable to consider the spans as partially loaded.

Generally speaking, *Case I.* rules the strains at each side of a point about $\frac{1}{3}$ the length of the two end spans, measured in either direction from the centre supports; *Case II.* gives maximum strains over the centre; and the unsymmetrical cases rule the dimensions of the two main spans, from where *Case I.* leaves off to the two ends respectively, and the strength of the latching or position of end supports in *IV.* and *V.* There may be some question whether it is wise to make the end reactions $= 0$ for the empty bridge in *Case II.*, the reason for which has been explained in discussing that case. By making them due to s, or still more, for *Case II.*, we decrease the strains over the centre, and, if certainty existed that the bridge would, before loading, be always supported properly at either end, and *Case II.* were then calculated for reactions at ends due to s, *Case V.*, which follows from the same arrangement, would have an advantage over *Case IV.* in producing less chord and web over the centre. If this certainty does not exist, and *II.* be calculated for end reactions equal to 0, *IV.* and *V.* are placed on an equal footing, as far as centre sections are concerned. A great point in favor of *IV.* is the little time and power required to latch the ends, as compared with that needed to lift them; though this last should not be overestimated. The power required to lift grows from 0, and can increase as high as half the weight of the girder, if the bridge be lifted off its centre supports.

We shall see, however, in a numerical example, that in point of fact comparatively little, sometimes no lift, is required to keep the bridge steady under the unsymmetrical load, according as the centre supports are pivoted or firm and snow on or off. In the numerical example, when pivoted and with snow off, it is necessary to have the end reactions due to a distributed load of about 4.5 tons per panel, or, roughly calculating for a 200 ft. draw end to end, about $\frac{3}{8}$ of 25. tons, or about 9.3 tons, and that only at the *end* of the lift.* By means of eccentrics, toggle-joints, or any other of the mechanisms that have *increasing* power, such a final pressure is easily produced. Such an increasing and rising pressure can also be produced by a *constant*, descending counter-weight. See "Zeitschrift für Bauwesen,"

* In the numerical calculations, $s = 4.8$ per panel, and $A = D$, when bridge is empty $= 2.9520 + 1.7712 + 3.1980 + 1.9188 = 9.84$. See table of reactions.

Berlin, 1871, plates 41 and 42, or also "Engineering," Vol. 12 (1871), page 154.*

In bridges subject to frequent opening and shutting, whatever arrangement is used at the ends, latch or lift, should be operated from the centre of the drawbridge to save time.

If *IV.* is to be used for railroad bridges, it must be carefully constructed, and the bolt that serves to latch the draw should be made to fit as exactly as possible in its socket, or all "lost motion" be taken up by the mechanism; if there be any play-room above or below, it would allow of appreciable destructive power being exercised respectively at the light end or over the centre every time an engine runs on the bridge.

A description of Mr. Macdonald's drawbridge at Point Street, Providence, R. I., 250 ft. long, and fitted with a latching apparatus, may be found in "Engineering," March 21, 1873, p. 202.

Whether the centre supports should, in the calculation, be considered as firm or pivoted (*a* or *b*), or the calculation made for both, the construction of the bridge must determine. Drawbridges will be differently constructed in this respect and in that of the manner of end supports, among other reasons, accordingly as they are more or less frequently opened. It would be an approximation, when the draw rested on the pivot, to consider it in effect as converted into a girder of two spans only, the pivot forming the middle support, and the framing over the pivot part of the truss, — but an approximation only, as may be proven. In a case of *symmetrical* loading of *AB* and *CD*, for example, (Fig. 10) the truss is evidently a case of *three*

Fig. 10.

continuous spans. If, now, the reactions *A* and *D* are calculated, they will be found *less* than if calculated supposing the truss to consist of *two* spans only, with exactly the same loading.

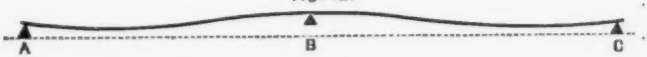
And the reason for this is not far to seek. The elastic line would take the shape as drawn in Fig. 11, rising higher than the level of the supports

Fig. 11.

* The same apparatus, together with the peculiar, and in some situations admirable, kind of drawbridge to which it is in this instance applied, is fully and clearly described and calculated by Häseler, Civil Engineer, in "Zeitschrift des Archt. and Ing Vereins zu Hannover," 1869, p. 412. The form of draw referred to is made without any "live ring," and the pivot is used as a support only when the bridge is being opened.

in the middle, or the case would be equivalent to the one of two spans, as sketched in Fig. 12, where the centre support is *higher* than the end supports;

Fig. 12.



but this, we *know*, decreases the end reactions, showing why, for a girder of three spans, the end reactions are less than for the same girder considered as composed of only two spans, but with the three supports on a level. Both cannot be right, and from the above three spans evidently is. Making the end reactions less involves making the centre ones greater, and the demonstration just given is as applicable for unsymmetrical loads as it is for symmetrical. To convert the girder into one of two spans only, it would be necessary to give it only one centre post, or arrangement of struts equivalent thereto, these bearing on the centre pivot, or on a *single* arm framing resting upon the same.

§ 6. Strains in the Girders. — It may seem strange to some readers that no word has yet been said about any thing but finding the *reactions* under the girder, as though that constituted the whole problem of the calculation of the strains in continuous drawbridges; and with but little exception it does. The reactions once found, any structure, of just the proper degree of strength to meet them and the loads that caused them, can be interpolated between the two, without the slightest difficulty or labor other than that inherent to the calculation of any framed structure, if the right way be only followed. And, as this has been written for the purpose of contributing to the elucidation of the construction, more especially, of *panel* drawbridges, the writer ventures to remark that he shall not consider his labors to have been in vain if, achieving nothing else, he will awaken in the minds of some of his readers a realizing sense of the beauty — that seems to be the correct appellation to apply to them — of the principles used by Aug. Ritter* in the calculation of all link structures, and which it is now intended to describe.

To understand this thoroughly, let us return to first principles and see what a framed link structure really is. It may be described as a properly designed skeleton system of lines, each line endowed with the faculty of exercising muscular or elastic force in both directions, each line pivoted

* See Ritter's book, cited in Appendix. In Van Nostrand's "Eclectic Engineering Magazine" for 1871, pp. 136 and 332, Ritter's method may be found translated and described for two examples, in a way, however, that to the author seems devoid of appropriate emphasis.

at its extremities, and then acted upon in *space* by generally only vertical forces, or, including roof-trusses, by vertical and horizontal forces, — in short, by forces in the plane of the structure; these forces are, in the first case, the reactions and loads, in the second the reactions, loads, and force of the wind. We have given the skeleton lines, the loads, and points of application, and the points of application of the reactions. We must find what has been called above the muscular force required to keep every thing in equilibrium, and the only way, in cases of continuous girders, to do this, is by first finding the reactions; and, going farther, there is much in favor of the statement that, when on account of there being only two supports, as in case of single spans, the laws of the lever (which, however, are only a special case of the elastic line) suffice, and strains, as this muscular force is more frequently called, can be found without starting from the reactions, the latter method is the best, simplest, and easiest remembered.

Taking now a girder as above described, acted upon by its forces (loads and reactions), and considering all parts in equilibrium, we have in any section:—

1. The sum of all the horizontal components each side the section must $= 0$, or $\Sigma H = 0$.

2. The sum of all the vertical components each side the section must $= 0$, or $\Sigma V = 0$.

3. The sum of all tendencies to rotate about any point in the plane of the girder — that is, all moments — must $= 0$, or $\Sigma M = 0$.

Ritter's simplification consists in using equation 3 *only*, which can always be done by merely *so choosing the points of rotation* that they shall be situated at the intersection of the direction of two out of three forces which are being investigated. The moments of two thus become 0, and there is left a simple equation for the value of the third. The general enunciation of the principle is this: Conceive a section of the girder, taken in such manner that it shall cut, if possible, only three members, and imagine the forces X, Y, and Z applied at the section, in the line of these members, and representing the strains in them. For that part of the structure which lies at either side of the section write the equation of moments, and so choose the point of rotation that, in determining X, the point of intersection of the directions Y and Z is taken; in determining Y, that of X and Z, and in determining Z that of X and Y.

For example, take a roof truss (Fig. 13):—

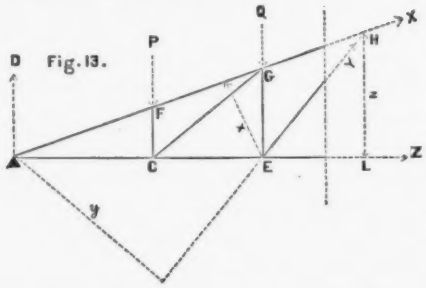


Fig. 13.

D = reaction, P and Q are loads, X, Y, Z strains required, x, y, and z lever arms. Moments with the motion of the hands of a clock, +; against it, —.

1. About E, intersection of Z and Y,
$$Xx - P.\overline{CE} + D.\overline{AE} = 0,$$
$$\text{or } X = \frac{P.\overline{CE} - D.\overline{AE}}{x}.$$

2. About A, intersection of X and Z,
$$-Yy + P.\overline{AC} + Q.\overline{AE} = 0,$$
$$\text{or } Y = \frac{P.\overline{AC} + Q.\overline{AE}}{y}.$$

3. About H, intersection of X and Y,
$$-Zz - Q.\overline{EL} - P.\overline{CL} + D.\overline{AL} = 0,$$
$$\text{or } Z = \frac{D.\overline{AL} - Q.\overline{EL} - P.\overline{CL}}{z}$$

Where a member cannot be cut by a section cutting only two others, it will always be possible (unless there is a redundance of members, and that means indeterminate strains and bad construction) to cut, if there are four together, some one of them, if five, two of them, &c., by a section taking in only three members; and, their strains once determined, they can be used as known forces in getting the others of the same group. Or, any number all meeting in one point and one more, may be cut, and the strain of this odd one determined; it being remembered that the section may be taken in any direction, straight or curvilinear. For trusses with parallel chords the demonstration is still correct; we have but to introduce ∞ as the distance at which the chords meet, and it will be found that the ∞s all cancel out, leaving easily remembered equations and forms of equations.

This constitutes, in fact, the whole method, which needs but to be tried to be appreciated. Its advantages are, primarily, that it throws aside all burdens upon the memory, in the shape of special rules for special cases, rules for each different kind of roof or bridge truss, not to mention a set of rules for variations of one and the same kind of girder.* The advantage of this it is difficult to overestimate; it exceeds, if any thing, the value of the well-known Napier's rules in Spherical Trigonometry. Another value lies in the readiness with which the strain on any one member can be calculated independently from that on any other; and, finally, in simple trusses, in being able to recognize at a glance, from the form of the equation for the strain on each member, what loads diminish and what loads increase the same; that is, under what loading it is maximum or minimum. The numerical examples have all been worked according to Ritter, and the reader is referred to them (and more especially to Ritter's book) for more on this branch of the subject. In the cases of finding the strains on the diagonals, it is frequently simpler to choose the first of the equations of equilibrium written above, instead of Ritter's method; that is, after having found the chord strains, by remembering that the horizontal components must equal 0, the diagonal strain becomes merely the horizontal component of the strain in one chord minus that of the strain in the other, multiplied by the secant of the angle the diagonal that connects them makes with the horizon, and it is of the kind of strain, tension or compression, according as needed for equilibrium. If the two chords are unlike, it always has the same quality, *t.* or *c.*, as the smallest; if both chords are alike (happens in Warren girder), the quality is opposite to that of the chords, and must equal their *sum* instead of difference.

Plate girders are not specially treated of in this paper; nothing new is to be said of their calculation, and, by getting equations for moments from the reactions, the curves of moments (and lines of shear from reactions) are easily drawn, and calculations of sizes of parts made as well known and long used. The value of plate girders for drawbridges is seriously diminished by the large surface they expose to the force of the wind. Cases are not wanting, where such bridges have been blown off the centre pier or have shut with such force as to break the centre pivot, &c. For large spans especially, their use must be decidedly condemned.

§ 7. Practical Considerations as to Choice of Form and System of Truss, Turn-tables, and Centre Pivots; and in Conclusion.—

* Most of these rules are, moreover, of the perplexing kind, sometimes known as "sink or swim" rules; *i.e.*, the answer sought is found in one of two ways, but you cannot tell which.

Laissle and Schuebler, in their work of 1870, give at the close a digest of 16 typical iron bridges that were built between 1855 and 1870, of all conceivable shapes and forms, both as regards chord, web, position of platform, number of trusses and corresponding number of tracks; two of them continuous over three spans; in fact no two alike, reducing them all to the same strain per unit of section, under the same load per unit of length. The result, if intended as an exhibition of the superiority of one style over another, is entirely nugatory, with the single exception that it is clearly better to support two tracks on two trusses only, than it is to support them on three or four. The authors arrive at the conclusion that the economy of each bridge depends, not on the choice of any one from among the dozens (hundreds it might almost be said) of good styles of skeleton outlines of chord and web that are fit to be used in any given locality, but on the skill of the constructor in each particular case; but this again is resolved, almost wholly, into that of reducing to a minimum that part of the weight of the bridge which is literally "dead" weight, — that is, carries no load, does not exercise muscular energy, and is only so much apoplectic, adipose tissue. Again, every member necessarily brings with it into the bridge more or less of this very dead weight; the less members, therefore, other things being equal, the better the bridge, the more load it will carry with the same total weight of material in it, or, conversely, the less material it will require to carry a given load. The rules for a choice of system for drawbridges will not be different from those for single fixed spans, with some few exceptions, due to reversal of strains from *c.* to *t.*, and *vice versa*.

At first sight, it might seem as though the height of the truss should be in a very different proportion to the span in draw from what it is in fixed bridges; because, when the draw is open, the span, of which each side represents half, is in fact double that of the doubly supported single span, of equal moment. It must not be forgotten, however, that when it is in this position the bridge is not loaded. Taken altogether, therefore, about the same proportions of height to span will obtain in continuous as in single spans, though with a leaning towards greater height in drawbridge spans than in single spans of equal length. In a recent competition for a draw 200 ft. over all and for two fixed spans each 121 ft. long, the same designer made only $660 difference in favor of 21 ft. trusses as against trusses 15 ft. high, on a bid for the latter height of $50,560. Perhaps Mr. Baker's * in-

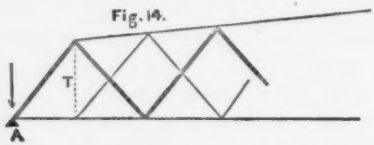
* On the Strengths of Beams, Columns, and Arches. B. Baker, London. E. and F. N. Spon, 1870.

vestigation as to the proper height of single span trusses and plate girders could be extended to the consideration of continuous spans also.

In continuous drawbridges, more than in single spans, the defects of separate counter-bracing become apparent. It is very much a question with the writer, whether separate counter braces and ties have not been used far too often in all structures, wooden or iron, but especially in the latter. A member that will resist both tension and compression can nearly always be made to weigh less than one to resist compression or tension running in one direction, added to its counter of like quality running opposite to it, especially when we have regard in the two cases to the useless dead weight above spoken of. In the way indicated, — that of introducing members capable of resisting both *t.* and *c.*, wherever they are liable to exchange of strain, — *any* kind of truss can be made to dispense with the customary counters and their extra weight.* This is recommended in all continuous spans, drawbridges included. In case of wooden trusses also, an iron rod parallel and beside the wooden strut, so as to make a counter-tie instead of counter-brace, would probably make a good wooden continuous truss.

A point to be careful about also is that of either superfluous or deficient members. The numerical example is one sent in in a recent competition for a drawbridge, slightly altered. As first designed, the end panels were as in Fig. 14, the dotted line representing a tie rod. Now when such a truss is

Fig. 14.

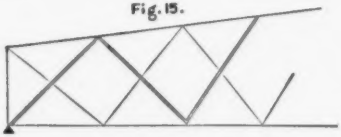


latched down, which is equivalent to putting a concentrated load on A, it will exert compression on the piece T. T cannot take, however, even the compression due to the loading on its own system, being only a rod, and should be made so as not to suffer from such compression, by having a link in it, or by being passed loosely through the top or bottom chord, &c. But, when thus cleared of compression, the minus reaction at A, which should

* It is curious sometimes to see counters introduced where, in point of fact, they appear like "cats in a strange garret." A "Post" truss, for example, is nothing but a Warren girder whose struts and ties are at an unequal angle with the horizon; and this web, instead of being made, like all other Warren girders, to take either *t.* or *c.* at the centre of the truss, is fitted out with counters in the shape of tension rods. They are, however, more proper here than in some other cases, owing to the great length of the regular tension members, which without them would have to be made to resist compression also.

pervade the whole webbing, can act on one system only, the one drawn in double lines, which must be characterized as undesirable. For this reason the end has been changed as in Fig. 15. Each system can now take any load

Fig. 15.



or reaction due to its own loading, and the end post belongs to the second system, the one drawn in *single* lines.*

A valuable paper on turn-tables and pivot centres by C. Shaler Smith will be found in the July number, 1874, of the "Transactions of the American Society of Civil Engineers." The data given there are probably unexcelled by those to be found scattered in the descriptions of various drawbridges. It is more such data that the profession is sadly in need of.

In the same number, Mr. Macdonald cites an equation from the ("Journal of the Society of German Engineers") Zeitschrift des Vereins Deutscher Ingenieure, Berlin, April, 1874, for the safe pressure on bearing rollers. The same subject may also be found analytically treated by Jules Gaudard, in "Études comparatives de Divers Systèmes de Ponts en fer," quoted in Spon's Dictionary of Civil Engineering, p. 789, and in Winkler's book, cited in the Appendix.

The writer makes no apology for the length of this paper or of any formulæ to be found in it. There is no "royal road" to the calculation of the strains in continuous girders, and the methods described in this paper are believed to be an improvement upon and are shorter than any of the exact modes of calculation that have come under the author's notice. It has been to him a pleasant task, though task it was. But there is no branch of engineering so cleanly cut and finely moulded in its results as this of finding the strains in framed structures. The reactions must first be equal in algebraic sum to that of the total loads; then, around any point of meeting of the several members in the structure the vertical components *up* must just balance those *down;* those to the *right* must balance those to the *left;* and any member cut in two must reveal in its interior an elastic or muscular force that will prevent revolution about *that* point

* There is still in this arrangement *some* indeterminateness; viz., the absolute direction that the reaction at *A* will take, whether up the end post or the first strut or what part on each. This is a defect common to all multiple systems of web, but is not of serious importance.

around which the parts of the structure *would* rotate, but for the duty by it performed. And the members and reactions must do all this, if need be, to any desired place of decimals, just as in the ledgers of some stately mercantile house the separate accounts and the trial balances must all foot up equally on both sides to the fraction of a cent. The principles involved in the one case are no more certain to produce the desired results than those upon which hinges the other; and, if those clear results are not forthcoming, it is but a case of human error or imperfection. The *principles* remain, perfect, immutable, and eternal.

PART II.

§ 8. Introductory to the Mathematical Investigations. — The arrangement of this Essay is intended to be such that, while the argument is all to be found in Part I., and its application illustrated in Part III., Part II. shall contain only the mathematical demonstrations, or the derivation of the formulæ presented for use, and used in Parts I. and III. The reader that is prepared to accept these formulæ on faith, therefore, need not read Part II. at all, but will, it is expected, find in the remainder of the paper sufficient to enable him to design any revolving drawbridge. Those that seek for mathematical proof will find the basis of Part II. in the work of Weyrauch, to which reference has many times been made; and the equations, taken from it, that are needed for the development of the formulæ it is now proposed to find, are the following: —

17. $$M_{r-1}\,l_{r-1} + 2\,M_r\,(l_{r-1}+l_r) + M_{r+1}\,l_r = -\left[\frac{c_{r-1}-c_r}{l_{r-1}} + \frac{c_{r+1}-c_r}{l_r}\right] 6\,EI$$
$$-\frac{1}{l_{r-1}}\sum_{r-1} Pa\,(l-a)\,(l+a) - \frac{1}{l_r}\sum_{r} Pa\,(l-a)\,(2\,l-a).$$

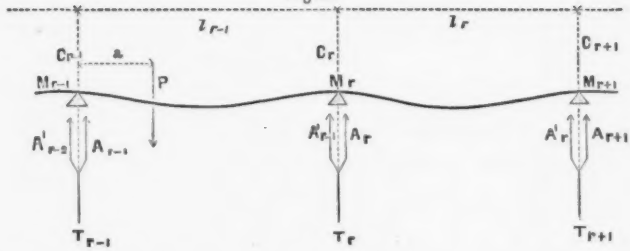
17a. $$M_{r-1}\,l_{r-1} + 2\,M_r\,(l_{r-1}+l_r) + M_{r+1}\,l_r = -\left[\frac{c_{r-1}-c_r}{l_{r-1}} + \frac{c_{r+1}-c_r}{l_r}\right] 6\,EI$$
$$-\tfrac{1}{4}\,q_{r-1}\,l_{r-1}^3 - \tfrac{1}{4}\,q_r\,l_r^3.$$

20. $$-V_0 = A = \frac{1}{l}\left[M' - M + \overset{1}{\Sigma}\,P\,(l-a)\right], \text{ and}$$

21. $$V_1 = A' = \frac{1}{l}\left[M - M' + \overset{1}{\Sigma}\,Pa\right],$$

in which the several characters have the meanings indicated by Fig. 16, and the schedule given below.

Fig. 16.



M_r, M_{r+1} &c., are moments over the supports, at left hand of the spans l_r, l_{r+1} &c.

M & M' are the moments over the left and right hand supports at either end of the span l.

l, l_r, l_{r+1} &c., are the lengths of spans.

c_r, c_{r+1} &c., are the ordinates of the points of support at left hand of spans l_r, l_{r+1} &c., referred to the axis of abscissæ.

E represents the modulus of elasticity.

I represents the moment of inertia of the beam, referred to its neutral axis.

P represents a load, or element of load, within the span l.

$\overset{l}{\Sigma} P$ indicates the summation of all loads P between 0 and l.

$\underset{r}{\Sigma} P$, $\underset{r+1}{\Sigma} P$, &c., indicate the summation of all the loads P, in the spans l_r, l_{r+1} &c.

a is the distance of point of application in the span l of the load P, from the left hand support.

q_r, q_{r+1} &c., are the amounts of uniformly distributed load, per unit of length, in the spans l_r, l_{r+1} &c.

V_0 & V_l are the vertical shearing forces at 0 and at l.

A & A' are the parts of the reactions at the left and right hand extremities of a span l, which are in equilibrium with the loads on that span.

$\overset{l}{\Sigma} P\,(l-a)$ means the summation of all the loads in the span l, each multiplied by the distance of its point of application from the right hand support; and

$\overset{l}{\Sigma} Pa$, the same sum multiplied by the distance of its point of application from the left hand support.

We pass now to the development of the general formulæ required for the calculation of pivot drawbridges.

§ 9. Equations for the Reactions of a Girder continuous over Three Supports. — Fig. 17. In **17**, above, make $r=2$, $r-1=1$, $r+1=3$,

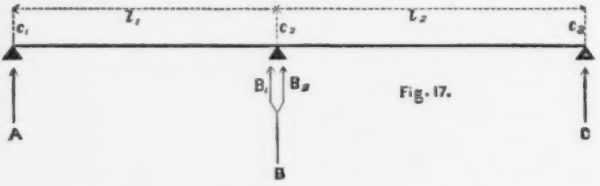


Fig. 17.

a in the span $l_1=el_1$, and a in the span $l_2=l_2-gl_2$. M_1 and M_3 are each $=0$, the ends being free (not walled in). Hence **17** becomes

$$2\,M_2\,(l_1+l_2)=-\left[\frac{c_1-c_2}{l_1}+\frac{c_3-c_2}{l_2}\right]6\,EI-\frac{1}{l_1}\,\Sigma\,Pe\,l_1\,(l_1^2-e^2\,l_1^2)$$
$$-\frac{1}{l_2}\,\Sigma\,Qgl_2\,(l_2^2-g^2\,l_2^2).$$

For convenience in writing, put the term $\left[\frac{c_1-c_2}{l_1}+\frac{c_3-c_2}{l_2}\right]6\,EI=Y$, and find the value of M_2:

28. $$M_2=\frac{-Y-\frac{1}{l_1}\,\Sigma\,Pe\,l_1\,(l_1^2-e^2\,l_1^2)-\frac{1}{l_2}\,\Sigma\,Qgl_2\,(l_2^2-g^2\,l_2^2)}{2\,(l_1+l_2)}.$$

Now, from **20** and **21**, introducing the new notation as just now in **17**, and making M_1 and $M_3=0$:

29. $$A=\frac{1}{l_1}\left[M_2+\Sigma\,P(l_1-el_1)\right].$$

30. $$B=B_1+B_2=\frac{1}{l_1}\left[-M_2+\Sigma\,Pel_1\right]+\frac{1}{l_2}\left[-M_2+\Sigma\,Qgl_2\right].$$

31. $$C=\frac{1}{l_2}\left[M_2+\Sigma\,Q\,(l_2-gl_2)\right].$$

Substituting the value of M_2 found above:

32. $$A=\frac{-Y-\frac{1}{l_1}\,\Sigma\,Pel_1\,(l_1^2-e^2\,l_1^2)-\frac{1}{l_2}\,\Sigma\,Qgl_2\,(l_2^2-g^2\,l_2^2)}{2\,l_1\,(l_1+l_2)}$$
$$+\frac{\Sigma\,P\,(l_1-el_1)}{l_1}.$$

33. $$B=\frac{Y+\frac{1}{l_1}\,\Sigma\,Pel_1\,(l_1^2-e^2\,l_2^2)+\frac{1}{l_2}\,\Sigma\,Qgl_2\,(l_2^2-g^2\,l_2^2)}{2\,l_1\,l_2}$$
$$+\frac{\Sigma\,P\,(l_1-el_1)}{l_1}+\frac{\Sigma\,Q\,(l_2-gl_2)}{l_2}.$$

34. $$C=\frac{-Y-\frac{1}{l_1}\Sigma Pel_1(l_1^2-e^2l_1^2)-\frac{1}{l_2}\Sigma Qgl_2(l_2^2-g^2l_2^2)}{2l_1(l_1+l_2)}$$
$$+\frac{\Sigma Q(l_2-gl_2)}{l_2}.$$

Generally $l_1=l_2=l$, and the points of support are on a level, in which last event Y vanishes. Then after a few changes we get:

35. $$A=\Sigma\left[\frac{P}{4}(4-e[5-e^2])\right]-\Sigma\left[\frac{Q}{4}g(1-g^2)\right],$$

36. $$B=\Sigma\left[\frac{P}{2}e(3-e^2)\right]+\Sigma\left[\frac{Q}{2}g(3-g^2)\right],$$

37. $$C=\Sigma\left[\frac{Q}{4}(4-g[5-g^2])\right]-\Sigma\left[\frac{P}{4}e(1-e^2)\right],$$

which are convenient forms for calculation. If further $e=g$ and $P=Q$ [equal spans, supports on a level, equal panel lengths and corresponding panel points equally loaded]:

38. $$A=C=\Sigma P\left[1-\frac{e(3-e^2)}{2}\right].$$

39. $$B=\Sigma P[e(3-e^2)].$$

For uniformly distributed loads, q_1 in span l_1 and q_2 in span l_2, for $Y=0$, *i.e.* the three supports on a level, $r-1=1$, $r=2$, and $r+1=3$, $M_1=M_3=0$ as before, **17***a* would become:

$$2M_2(l_1+l_2)=-\tfrac{1}{4}q_1l_1^3-\tfrac{1}{4}q_2l_2^3.$$

Similarly we should have for values of A, B, and C, derived from **20** and **21**, substituting

$$\int_0^{l_1} q_1\,dx\,(l_1-x) \text{ for } \Sigma P(l-a) \text{ in span } l_1,$$

and $$\int_0^{l_2} q_2\,dx.\,x \text{ for the same in span } l_2,$$

also $$\int_0^{l_1} q_1\,dx.\,x \text{ for } \Sigma Pa \text{ in span } l_1,$$

and $$\int_0^{l_2} q_2\,dx\,(l_2-x) \text{ for the same in span } l_2:$$

40. $$A=\frac{1}{l_1}\left[M_2+\tfrac{1}{2}q_1l_1^2\right].$$

41. $B = \frac{1}{l_1}\left[-M_2 + \frac{1}{2}q_1 l_1^2\right] + \frac{1}{l_2}\left[-M_2 + \frac{1}{2}q_2 l_2^2\right]$.

42. $C = \frac{1}{l_2}\left[M_2 + \frac{1}{2}q_2 l_2^3\right]$.

Or substituting value of M_2:

43. $A = \frac{-\frac{1}{4}(q_1 l_1^3 + q_2 l_2^3)}{2(l_1 + l_2) l_1} + \frac{1}{2}q_1 l_1$.

44. $B = \frac{\frac{1}{4}(q_1 l_1^3 + q_2 l_2^3)}{2 l_1 l_2} + \frac{1}{2}(q_1 l_1 + q_2 l_2)$.

45. $C = \frac{-\frac{1}{4}(q_1 l_1^3 + q_2 l_2^3)}{2(l_1 + l_2) l_2} + \frac{1}{2}q_2 l_2$.

These are sometimes found in the forms

43a. $A = \frac{3 q_1 l_1^3 + 4 q_1 l_1^2 l_2 - q_2 l_2^3}{8 l_1 (l_1 + l_2)}$.

44a. $B = \frac{q_1 l_1^4 + 5 q_1 l_1^3 l_2 + 4 q_1 l_1^2 l_2^2 + 4 q_2 l_1^2 l_2^2 + 5 q_2 l_1 l_2^3 + q_2 l_2^4}{8 l_1 l_2 (l_1 + l_2)}$.

45a. $C = \frac{3 q_2 l_2^3 + 4 q_2 l_1 l_2^2 - q_1 l_1^3}{8 l_2 (l_1 + l_2)}$.

If in these $l_1 = l_2 = l$,

46. $A = \frac{l}{16}(7 q_1 - q_2)$.

47. $B = \frac{10 l}{16}(q_1 + q_2)$.

48. $C = \frac{l}{16}(7 q_2 - q_1)$.

Finally for $q_1 = q_2$ (equal spans, supports on a level, both spans equally and uniformly loaded):

49. $A = C = \frac{3}{8} ql$.

50. $B = \frac{10}{8} ql$.

§ 10. Equations for the Reactions of a Girder continuous over

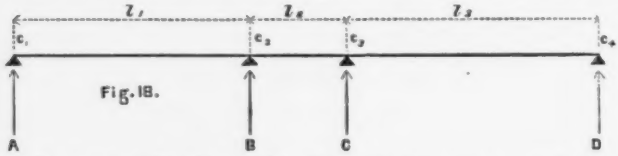


Fig. 18.

four Supports. — Fig. 18. Making in **17**, r first $= 2$ and then $= 3$, first M_1 and again M_4 become $= 0$, and the equation becomes

$2M_2(l_1+l_2)+M_3 l_2 = -Y_1 - \frac{1}{l_1}\Sigma Px(l_1^2-x_2) - \frac{1}{l_2}\Sigma Qy(l_2-y)(2l_2-y)$, and

$M_2 l_2 + 2M_3(l_2+l_3) = -Y_2 - \frac{1}{l_2}\Sigma Qy(l_2^2-y^2) - \frac{1}{l_3}\Sigma Rz(l_3-z)(2l_3-z)$,

designating for brevity $\left[\frac{c_1-c_2}{l_1}+\frac{c_2-c_2}{l_2}\right] 6EI$ by Y_1

and $\left[\frac{c_2-c_3}{l_2}+\frac{c_4-c_3}{l_3}\right] 6EI$ by Y_2, replacing also the general load P and distance a by P, Q, R and x, y, z in the spans l_1, l_2, l_3, respectively. Solving these equations for M_2 and M_3, we obtain :

51. $$M_2 = \frac{\left[Y_1+\frac{1}{l_1}\Sigma Px(l_1^2+x^2)+\frac{1}{l_2}\Sigma Qy(l_2-y)(2l_2-y)\right]2(l_2+l_3)}{l_2^2-4(l_2+l_3)(l_1+l_2)} - \frac{\left[Y_2+\frac{1}{l_2}\Sigma Qy(l_2^2-y^2)+\frac{1}{l_3}\Sigma Rz(l_3-z)(2l_3-z)\right]l_2}{l_2^2-4(l_2+l_3)(l_1+l_2)}.$$

52. $$M_3 = \frac{\left[Y_2+\frac{1}{l_2}\Sigma Qy(l_2^2-y^2)+\frac{1}{l_3}\Sigma Rz(l_3-z)(2l_3-z)\right]2(l_1+l_2)}{l_2^2-4(l_2+l_3)(l_1+l_2)} - \frac{\left[Y_1+\frac{1}{l_1}\Sigma Px(l_1^2-y^2)+\frac{1}{l_2}\Sigma Qy(l_2-y)(2l_2-y)\right]l_2}{l_2^2-4(l_2+l_3)(l_1+l_2)}.$$

Comparing these, we find that the first quantity enclosed in [] in the value of M_2 is entirely similar with the second quantity enclosed in [], in the value of M_3, and that the same similarity occurs between the second term of M_2 and the first of M_3, so that for convenience we may write,

53. $$M_2 = \frac{2(l_2+l_3)V - l_2 W}{U}.$$

54. $$M_3 = \frac{2(l_1+l_2)W - l_2 V}{U}.$$

From **20** and **21**, making $M_1 = M_4 = 0$, we obtain

55. $$A = \frac{1}{l_1}\left[M_2+\Sigma P(l_1-x)\right].$$

56. $$B = \frac{1}{l_1}\left[-M_2+\Sigma Px\right]+\frac{1}{l_2}\left[-M_2+M_3+\Sigma Q(l_2-y)\right].$$

57. $$C = \frac{1}{l_2}\left[M_2-M_3+\Sigma Qy\right]+\frac{1}{l_3}\left[-M_3+\Sigma R(l_3-z)\right].$$

58. $$D = \frac{1}{l_3}\left[M_3+\Sigma Rz\right].$$

Substituting the values of M_2 and M_3 given above, and reducing, these become:

59. $$A = \frac{2(l_2+l_3)V - l_2 W}{l_1 U} + \frac{\Sigma P(l_1-x)}{l_1}.$$

60. $$B = \frac{(2l_1+l_2)(l_1+l_2)W - [2(l_1+l_2)(l_2+l_3)+l_1 l_2]V}{l_1 l_2 U} + \frac{\Sigma Px}{l_1} + \frac{\Sigma Q(l_3-y)}{l_2}.$$

61. $$C = \frac{(2l_3+l_2)(l_2+l_3)V - [2(l_1+l_2)(l_2+l_3)+l_2 l_3]W}{l_2 l_3 U} + \frac{\Sigma Qy}{l_2} + \frac{\Sigma R(l_3-z)}{l_3}.$$

62. $$D = \frac{2(l_1+l_2)W - l_2 V}{l_3 U} + \frac{\Sigma Rz}{l_3}.$$

Usually Y_1 and Y_2 will $=0$, the supports being on the same level, $l_1=l_3=l$, $l_2=ml$, $x=el$, $l_3-z=gl$, and $Q=0$. Under these conditions

$$V = \Sigma Pel^2(1-e^2),$$
$$W = \Sigma Rgl^2(1-g^2),$$
$$U = l^2[m^2-4(1+m^2)], \text{ and}$$

63. $$A = \Sigma P(1-e)\left[1 - \frac{2e(1+m)(1+e)}{3m^2+8m+4}\right] + \Sigma R(1-g)\frac{mg(1+g)}{3m^2+8m+4}.$$

64. $$B = \Sigma Pe\left[1 + \frac{(2+5m+2m^2)(1-e^2)}{m(3m^2+8m+4)}\right] - \Sigma Rg\frac{(2+3m+m^2)(1-g^2)}{m(3m^2+8m+4)}.$$

65. $$C = \Sigma Rg\left[1 + \frac{(2+5m+2m^2)(1-g^2)}{m(3m^2+8m+4)}\right] - \Sigma Pe\frac{(2+3m+m^2)(1-e^2)}{m(3m^2+8m+4)}.$$

66. $$D = \Sigma R(1-g)\left[1 - \frac{2g(1+m)(1+g)}{3m^2+8m+4}\right] + \Sigma P(1-e)\frac{me(1+e)}{3m^2+8m+4}.$$

If further the loading be symmetrical, that is, if $e = g$ and $P = R$, a further reduction is possible:

67. $A = D = \Sigma P (1 - e) \left[1 - \frac{e (m+2) (1+e)}{3 m^2 + 8 m + 4}\right]$.

68. $B = C = \Sigma P e \left[1 + \frac{(m+2) (1 - e^2)}{3 m^2 + 8 m + 4}\right]$.

Similarly for uniformly distributed loading, ΣP will become $\int_0^{l_1} q_1 \, dx$, $\Sigma Q, \int_0^{l_2} q_2 \, dx$, $\Sigma R, \int_0^{l_3} q_3 \, dx$, and we shall have from formulæ **59–62**:

69. $A = \frac{2 (l_2 + l_3) [\frac{1}{4} q_1 l_1^3 + \frac{1}{4} q_2 l_2^3] - l_2 [\frac{1}{4} q_2 l_2^3 + \frac{1}{4} q_3 l_3^3]}{l_1 [l_2^2 - 4 (l_1 + l_2) (l_2 + l_3)} + \frac{1}{2} q_1 l_1$.

70. $B = \frac{(l_1 + l_2) (2 l_1 + l_2) [\frac{1}{4} q_2 l_2^3 + \frac{1}{4} q_3 l_3^3]}{l_1 l_2 (l_2^2 - 4 (l_1 + l_2) (l_2 + l_3)]}$

$$- \frac{[l_1 l_2 + 2 (l_1 + l_2) (l_2 + l_3)] (\frac{1}{4} q_1 l_1^3 + \frac{1}{4} q_2 l_2^3)}{l_1 l_2 [l_2^2 - 4 (l_1 + l_2) (l_2 + l_3)]}.$$

71. $C = \frac{(l_3 + l_2) (2 l_3 + l_2) [\frac{1}{4} q_1 l_1^3 + \frac{1}{4} q_2 l_2^3]}{l_1 l_2 [l_2^2 - 4 (l_1 + l_2) (l_2 + l_3)]}$

$$- \frac{[l_1 l_2 + 2 (l_1 + l_2) (l_2 + l_3)] (\frac{1}{4} q_2 l_2^3 + \frac{1}{4} q_3 l_3^3)}{l_1 l_2 [l_2^2 - 4 (l_1 + l_2) (l_2 + l_3)]}.$$

72. $D = \frac{2 (l_1 + l_2) [\frac{1}{4} q_2 l_2^3 + \frac{1}{4} q_3 l_3^3] - l_2 [\frac{1}{4} q_1 l_1^3 + \frac{1}{4} q_2 l_2^3]}{l_3 [l_2^2 - 4 (l_1 + l_2) (l_2 + l_3)]} + \frac{1}{2} q_3 l_3$.

When again $l_1 = l_3 = l$, $l_2 = ml$ (m for drawbridges would be a proper fraction), also $q_1 = q_3 = q$:

73. $A = D = \frac{1}{2} ql \left[1 - \frac{2 + m}{2 (3 m^2 + 8 m + 4)}\right] - \frac{1}{2} q_2 l_2 \frac{m^3 (2 + m)}{2 (3 m^2 + 8 m + 4)}$.

74. $B = C = \frac{1}{2} ql \left[1 + \frac{2 + m}{2 (3 m^2 + 8 m + 4)}\right]$

$$+ \frac{1}{2} q_2 lm \left[1 + \frac{m^2 (2 + m)}{2 (3 m^2 + 8 m + 4)}\right].$$

Finally when $q_2 = q$ and $m = 1$, that is $l_1 = l_2 = l_3 = l$,

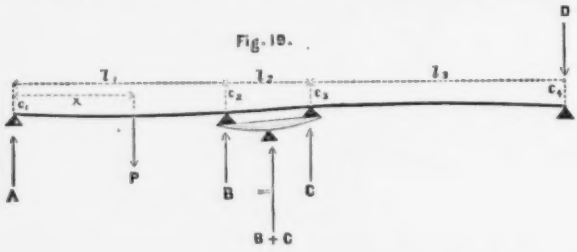
75. $A = D = \frac{4}{10} ql$,

76. $B = C = \frac{11}{10} ql$.

There remain to be developed the equations for the very peculiar cases, when two of the supports are themselves supported on the ends of an inflex-

ible body, which, in turn, rests upon a knife edge or pivot at its centre. (Fig. 19.) It is evident that, for equilibrium, the reactions of such supported

Fig. 19.



supports must be equal. An incidental quality is that, when such supports are *not* symmetrically loaded, the ordinate of one support, measured from the line connecting them when in a horizontal or symmetrically loaded position, equals the ordinate of the other measured in the opposite direction, or, for a three span girder, $c_2 = -c_3$.* The calculation of the equations for the reactions under these conditions is tedious but not difficult, after the method to be followed and the circumstances to be taken into account are once clearly imagined.

§ 11. Equations for the Reactions of a Girder continuous over Four Supports, the Two Centre Ones resting upon an Inflexible Body, which is pivoted at the Centre. — For our cases of unsymmetrical loading, we must make $Q = R = 0$. We then have the main condition of equilibrium, that $B = C$ (see fig. and equations in § 10).

77. $$B = \frac{1}{l_1}\left[-M_2 + \Sigma Px\right] + \frac{1}{l_2}\left[M_3 - M_2\right] = \frac{1}{l_2}\left[M_2 - M_3\right] - \frac{M_3}{l_3} = C,$$

or
$$-\frac{M_2}{l_1} - \frac{2M_2}{l_2} + \frac{2M_3}{l_2} + \frac{M_3}{l_3} = -\frac{\Sigma Px}{l_1} = T;$$

also,

78. $$2M_2(l_1 + l_2) + M_3 l_2 = -Y_1 - \frac{1}{l_1}\Sigma Px(l_1^2 - x^2) = R.$$

79. $$M_2 l_2 + 2M_3(l_2 + l_3) = -Y_2 = S.$$

80. From **79**, $M_2 = \dfrac{S - 2M_3(l_2 + l_3)}{l_2}$. Substituting in **78** and finding value of M_3, —

* This is exactly true when the pivot is situated in the line joining B and C, and equi-distant from these points. There is no difficulty about developing the equations when $c_2 = -nc_3$, or even $f(c_2) = -f(c_3)$.

81. $M_2 = \dfrac{l_2 R - 2(l_1+l_2) S}{l_2^2 - 4(l_1+l_2)(l_2+l_3)}$. Substituting **80** in **77** will give

82. $M_2 = \dfrac{l_1 l_2^2 l_3 T + (l_2 l_3 + 2 l_1 l_3) S}{2 l_2^2 l_3 + 2 l_2 l_3^2 + 4 l_1 l_2 l_3 + 4 l_1 l_3^2 + l_1 l_2^2}$.

Make now $l_1 = l_3 = l$, and $l_2 = ml$, and put **81** = **82**, we get, —

83. $S - R = ml^2 T$. Substituting values of S, R, and T, we get, finally,

84. $Y_2 - Y_1 = \dfrac{1}{l} \Sigma Px (l^2 - x^2 + ml^2)$.

But $\quad -Y_1 = -\left[\dfrac{c_1 - c_2}{l_1} + \dfrac{c_3 - c_2}{l_2}\right] 6 EI,$

and $\quad -Y_2 = -\left[\dfrac{c_2 - c_3}{l_2} + \dfrac{c_4 - c_3}{l_3}\right] 6 EI$, also $c_1 = c_4 = 0$, and $c_2 = -c_3$;

$$\therefore Y_1 = -\left[\frac{2c + mc}{ml}\right] 6 EI, \text{ and } Y_2 = \left[\frac{2c + mc}{ml}\right] 6 EI, \text{ and}$$

85. $-Y_1 = \dfrac{1}{2l} \Sigma Px (l^2 - x^2 + ml^2)$

86. $-Y_2 = -\dfrac{1}{2l} \Sigma Px (l^2 - x^2 + ml^2)$.*

Substituting values of $-Y_1$ and $-Y_2$ in **78** and **79**, and reducing, we get, —

87. $M_2 = \dfrac{-\left(\dfrac{2+m}{2l^2}\right) \Sigma Px (l^2 - x^2) + \dfrac{(2+3m)\, m}{2} \Sigma Px}{3m^2 + 8m + 4}$.

88. $M_3 = \dfrac{-\left(\dfrac{2+m}{2l^2}\right) \Sigma Px (l^2 - x^2) - \dfrac{(2+3m)\, m}{2} \Sigma Px}{3m^2 + 8m + 4}$.

These values substituted in the general equations for A, B, C, and D **(55–58)** [$Q = R = 0$, $l_1 = l_3 = l$, $l_2 = ml$] give finally, —

89. $A = \dfrac{-(2+m) \Sigma Px (l^2 - x^2) + (2+3m) ml^2 \Sigma Px}{2l^3 (3m^2 + 8m + 4)} + \dfrac{1}{l} \Sigma P(l - x)$.

90. $B = C = \dfrac{(2+m) \Sigma Px (l^2 - x^2)}{2l^3 (3m^2 + 8m + 4)} + \dfrac{\Sigma Px}{2l}$.

91. $D = -\dfrac{(2+m) \Sigma Px (l^2 - x^2) + (2+3m) ml^2 \Sigma Px}{2l^3 (3m^2 + 8m + 4)}$,

and $A + B + C + D = \Sigma P$, as they should.

To bring these equations to the same known quantities as equations **63–66**, we have but to substitute el for x.

---

* From the values of Y_1 and Y_2, in terms of c, m, and l, and the same values in terms of ΣPx, m, and l, the value of c can readily be found, but it will contain the term EI.

§ 12. Equations for the Reactions of a Girder continuous over Three Supports, the Two Right Hand Ones resting upon an Inflexible Body, which is pivoted at the Centre. — Taking again for our case of unsymmetrical loading $Q=0$, $l_1=l$, $l_2=ml$, we have for the main condition of equilibrium $B=C$, or

$$\frac{1}{l}\Big[-M_2+\Sigma Px\Big]+\frac{1}{ml}\Big[-M_2\Big]-\frac{1}{ml}\Big[M_2\Big]=0, \text{ or}$$

92. $M_2=\dfrac{m\,\Sigma Px}{m+2}$. Also,

93. $2M_2(l+ml)=-Y-\dfrac{1}{l}\Sigma Px(l^2-x^2)$, or

94. $M_2=\dfrac{-Y-\dfrac{1}{l}\Sigma Px(l^2-x^2)}{2l(1+m)}$. From **92** and **93**,

95. $-Y=\dfrac{2l(1+m)\,m\,\Sigma Px+\dfrac{m+2}{l}\Sigma Px(l^2-x^2)}{m+2}$.

Substituting in **93**,

96. $M_2=\dfrac{m}{m+2}\Sigma Px$,* and substituting this in the general equations for A, B, and C (**29**–**31**).

97. $A=\dfrac{m}{l(m+2)}\Sigma Px+\dfrac{\Sigma P}{l}(l-x)$.

98. $B=C=\dfrac{\Sigma Px}{l(m+2)}$,

and $A+B+C$ is, as it should be, equal to ΣP.

Again, substituting el for x, we obtain A, B, and C, in terms of the same known quantities as in equations **35**–**37**.

§ 13. In a Girder continuous over Four Level Supports, the Two Centre Ones Pivoted (same as in § 11), to find what Loading uniformly distributed, added to a given Loading uniformly distributed over the Span AB, will make $D=0$. — We had for the unsymmetrical load (**91**), —

$$D=-\frac{(2+m)\,\Sigma Px(l^2-x^2)+(2+3m)\,ml^2\,\Sigma Px}{2l^3(3m^2+8m+4)}.$$

* Being the same as found (**92**), which shows that the principles of the lever alone suffice in this instance, and that these principles are but a special case of those of the elastic line.

For uniform loading over the whole span AB, we must substitute $\int_0^l q_1\,dx$ for ΣP, which gives,—

99. $$D=-\frac{\frac{(2+m)}{4}\,q_1\,l^4+(2+3\,m)\,mq_1\frac{l^4}{2}}{2\,l^4\,(3\,m^2+8\,m+4)}=\text{unsymmetrical } D.$$

From **70**, with $q_2=q$, we have,—

100. $$D=\tfrac{1}{2}\,ql\left[1-\frac{(2+m)}{2\,(3m^2+8m+4)}\right]-\tfrac{1}{2}\,ql\,\frac{m^3\,(2+m)}{2\,(3m^2+8\,m+4)}=\text{syml. } D.$$

The condition is that unsymmetrical $D+$ symmetrical D shall equal 0, or

$$-\frac{\frac{(2+m)}{4}\,q_1\,l^4+(2+3\,m)\,mq_1\frac{l^4}{2}}{2\,l^3\,(3\,m^2+8\,m+4)}=\tfrac{1}{2}\,ql\left[1-\frac{(2+m)}{2\,(3\,m^2+8\,m+4)}\right]$$
$$-\tfrac{1}{2}\,ql\,\frac{m^3\,(2+m)}{2\,(3\,m^2+8\,m+4)},$$

from which is found,

101. $$q=\frac{q_1\,(2+5\,m+6m^2)}{2\,(6+15\,m+6\,m^2-2\,m^3-m^4)}=\text{uniform load per unit of length over all three spans, necessary to make } D=0,$$ when span AB is loaded with q_1 per unit of length in addition to q, and the supports are in the condition above stated.

§ 14. In a Girder continuous over Two Unequal Spans, to find what Loading uniformly distributed, added to a given Load uniformly distributed over the Span AB, will make $C=0$.— We had (**45***a*),—

$$C=\frac{3\,q_2\,l_2^3+4\,q_2\,l_1\,l_2^2-q_1\,l_1^3}{8\,l_2\,(l_1+l_2)}.$$

For $q_2=0$, or unsymmetrical loading, and $l_2=ml$, we have

102. Unsymmetrical $$C=-\frac{q_1\,l}{8\,m\,(1+m)}.$$

For $q_1=q_2=q$, or symmetrical loading,

103. Symmetrical $$C=\frac{3\,ql\,m^3+4\,ql\,m^2-ql}{8\,m\,(1+m)},$$ or putting

$$-\frac{q_1\,l}{8\,m\,(1+m)}+\frac{3\,ql\,m^3+4\,ql\,m^2-ql}{8\,m\,(1+m)}=0,$$ we obtain,

104. $$q=\frac{q_1}{3\,m^3+4\,m^2-1}.$$

PART III.

§ 15. Numerical Example. (See Plate I.) — *Loading.* The assumed dead load of each truss (which is in 3 spans of 90, 20, and 90 feet, and made continuous over them all) is taken at 1000 lbs., the snow load at 200 lbs., and the live load at 1400 lbs. per lineal foot. Each panel is 18 feet long, except the centre one, which is 20 feet long. The above loads, concentrated at the panel points and points of support, are as follows, in tons:—

Over each end support . . . . { 4.5 tons dead load. / 0.9 tons snow ,, / 6.3 tons live ,,

At each panel point . . . . . { 9.0 tons dead load. / 1.8 tons snow ,, / 12.6 tons live ,,

Over each centre support . . . { 9.5 tons dead load. / 1.9 tons snow ,, / 13.3 tons live ,,

Lever Arms* for the First System (in feet).

$U_{1\&2}$	19.929	L_1	18.5	S_1	172.01	T_2	184.02
$U_{3\&4}$	22.02	$L_{2\&3}$	21.5	S_3	211.62	T_4	222.43
U_5	25.909	$L_{4\&5}$	24.5	S_5	251.44	P_2	∞
U_6	26.	L_6	26.	P_1	222.	T_6	$\infty \sin \phi$

Lever Arms* for the Second System (in feet).

$U_{2\&3}$	21.425	$L_{1\&2}$	20.	S_2	191.77	T_1	165.01
$U_{4\&5}$	24.414	$L_{3\&4}$	23.	S_4	231.52	T_3	203.17
U_6	26.	$L_5 \& L_6$	26.			T_5	241.72

Formulæ for the Reactions. — For our case $m = \frac{2}{9}$. Substituting this in the equations for reactions, and making $e = g = k$, they reduce to the following:—

* See Plate II.

Formulæ for four level supports; spans AB and CD symmetrically loaded, from **67** and **68**,

$$\begin{cases} A = D = \frac{\Sigma P}{8}(8 - 11k + 3k^3). \\ B = C = \frac{\Sigma P}{8}(11k - 3k^3). \end{cases}$$

Formulæ for four level supports; span AB only loaded, from **63–66**,

$$\begin{cases} A = \frac{\Sigma P}{80}(80 - 113k + 33k^3). \\ B = \frac{\Sigma P}{16}(55k - 39k^3). \\ C = -\frac{33}{16}\Sigma P(k - k^3). \\ D = \frac{3}{80}\Sigma P(k - k^3). \end{cases}$$

Formulæ for four supports, when B and C are "pivoted" and AB only loaded, from **89–91**,

$$\begin{cases} A = \frac{\Sigma P}{80}(80 - 91k + 15k^3). \\ B = C = \frac{\Sigma P}{16}(11k - 3k^3). \\ D = -\frac{\Sigma P}{80}(19k - 15k^3). \end{cases}$$

Formulæ for three supports, when B and C are "pivoted" and AB only loaded, from **97–98**,

$$\begin{cases} A = \frac{\Sigma P}{10}(10 - 9k). \\ B = C = \frac{9}{20}\Sigma Pk. \end{cases}$$

In all cases $A + B + C + D = \Sigma P$, except the first written, when $A + B + C + D = 2\Sigma P$, as it should.

Example. — Snow load in the first system :—

$P = 1.8$ & $k = .2$; $P = 1.8$ & $k = .6$; $P = 1.9$ & $k = 1$

$\therefore$ by first formula, $A = D = 1.8 \times .984 = 1.7712$

$B = C = 1.8 \times 1.016 + 1.9 = 3.7288$

Snow load in the second system :—

$P = 0.9$ & $k = 0$; $P = 1.8$ & $k = .4$; $P = 1.8$ & $k = .8$

$\therefore$ by same formula as above :

$A = D = 1.8 \times .566 + 0.9 = 1.9188$

$B = C = 1.8 \times 1.434 = 2.5812$

The reactions of the girder, for the several cases given in Part I., and found in a manner similar to the above, are given in the following Table :—

REACTIONS (IN TONS).

Manner of Support.	Load.	FIRST SYSTEM.				SECOND SYSTEM.			
		A	*B*	*C*	*D*	*A*	*B*	*C*	*D*
Case I.	Dead,	0.	27.5	27.5	0.	0.	22.5	22.5	0.
	Snow,	0.	5.5	5.5	0.	0.	4.5	4 5	0.
		0.	33.	33.	0.	0.	27.	27.	0.
Case II.	Dead,	0.	27.5	27.5	0.	0.	22.5	22.5	0.
	Snow,	1.7712	3.7288	3.7288	1 7712	1 9188	2 5812	2.5812	1.9188
	Live,	12.3984	26.1016	26.1016	12.3984	13.4316	18.0684	18.0684	13 4316
		14.1696	57.3304	57.3304	14.1696	15.3504	43.1496	43.1496	15.3504
Case III.(*a*)	Dead,	0.	27.5	27.5	0.	0.	22.5	22.5	0.
	Snow,	1.77120	3.7288	3.7288	1.77120	1.91880	2.5812	2 5812	1.91880
	Live,	12.12624	41.0704	−14.9688	0.27216	13.13676	34.2846	−16.2162	0.29484
		13.89744	72.2992	16.2600	2.04336	15.05556	59.3658	8.8650	2.21364
Case III.(*b*)	Dead,	0.	27.5	27.5	0.	0.	22.5	22.5	0.
	Snow,	0.	5.5	5.5	0.	0.	4 5	4.5	0.
	Live,	17.458	10.521	10.521	0.	17.892	6.804	6.804	0.
		17.458	43.521	43.521	0.	17.892	33.804	33.804	0.
Case IV.(*a*)		Same reactions	as for *III.*(*a*)	[because D=+]					
Case IV.(*b*)	Dead,	0.	27.5	27.5	0.	0.	22.5	22.5	0.
	Snow,	1.7712	3.7288	3.7288	1.7712	1.9188	2.5812	2.5812	1.9188
	Live,	14.9282	13.0508	13.0508	−2.5298	15.6618	9.0342	9.0342	−2.2302
		16.6994	44 2796	44.2796	−0.7586	17.5806	34.1154	34.1154	−0.3114
Case V.(*a*)	Dead,	0.	18.33333+	18.33333+	0.	0.	15.	15.	0.
	Dead,	2.95200	6.21466+	6.21466+	2.95200	3.19800	4.3020	4.3020	3.19800
	Snow,	1.77120	3.72880	3.72880	1.77120	1.91880	2.5812	2.5812	1.91880
	Live,	12.12624	41.07040	−14.96880	0.27216	13.13676	34 2846	−16.2162	0.29484
		16.84944	69.3472	13.308	4.99536	18.25356	56.1678	5 067	5.41164
Case V.(*b*)	Dead,	0.	18.33333+	18.33333+	0.	0.	15.	15	0.
	Dead,	2.9520	6.21466+	6.21466+	2.9520	3.1980	4.3020	4.3020	3.1980
	Snow,	1.7712	3.72880	3.72880	1.7712	1.9188	2.5812	2.5812	1.9188
	Live,	14.9282	13.65080	13.05080	−2.5298	15.6618	9.0342	9.0342	−2.2302
		19.6514	41.3276	41.3276	2,1934	20.7786	30.9174	30.9174	2,8866

Strain Sheets.—(See Plates III.–IX.) The figures represent resultant strains in tons of 2,000 lbs. (+) means tension, (—) means compression. To abbreviate calculation, the dead load has been taken as though it were concentrated at the lower panel points; it is more exact, and in principle just as easy to follow out, to take the own weight as divided into two main parts, (1) that of the truss, and (2) that of the roadway platform, and each as though applied at that part of the skeleton outline of the bridge at which it *is* applied. A close approximation is to take the platform weight + ½ weight of truss, at lower panel points, and ½ weight of truss at upper panel points in "through" bridges; reverse this arrangement in "deck" bridges, and other combinations will readily suggest themselves in their proper places.

To illustrate the manner in which these Strain Sheets have been calculated, the computation for *Case II.* is next given, worked throughout *strictly* according to Ritter's method, although for chord members another way may sometimes be shorter, as alluded to in Part I.

Equations for finding the Strains in the several members of the Draw, loaded and supported (that is, reacted on) as in Case II.—By making the proper substitutions for loads and reactions, all the other cases can be calculated according to the same formulæ.* The centre panel diagonals are made to take tension only; when the resultant strain on one of them comes out *minus* therefore, neglect that diagonal, and make the calculation over again [for that and for such other members as depend upon such change], supposing now only the other diagonal in place. All other members are made to take either the compression or extension that they may be called upon to withstand.

FIRST SYSTEM.

$$
\begin{aligned}
U_{1\,\&\,2} &\times 19.929 + 14.17 \times 18 = 0.\\
U_{3\,\&\,4} &\times 22.92 \ + 14.17 \times (3 \times 18) - 23.4 \times (2 \times 18) = 0.\\
U_5 &\times 25.909 + 14.17 \times (5 \times 18) - 23.4 \times [(2+4) \times 18] = 0.\\
U_6 &\times 26. \quad + 14.17 \times [(5 \times 18) + 20] + (57.33 - 24.7) \times 20\\
&\quad - 23.4 \times [((4 \times 18) + 20) + ((2 \times 18) + 20)] = 0.
\end{aligned}
$$

$$
\begin{aligned}
-L_1 &\times 18.5 + 14.17 \times 0 = 0.\\
-L_{2\,\&\,3} &\times 21.5 + 14.17 \times (2 \times 18) - 23.4 \times 18 = 0.\\
-L_{4\,\&\,5} &\times 24.5 + 14.17 \times (4 \times 18) - 23.4 \times [(1+3) \times 18] = 0.\\
-L_6 &\times 26. \ + 14.17 \times (5 \times 18) - 23.4 \times [(2+4) \times 18] = 0.
\end{aligned}
$$

* The several equations have been written out in full, and without the slightest attempt at reduction; on the contrary, *all the elements of each term* have been conspicuously written and separated, so as to enable any one to follow the process of evolving the equations for each member in any case. In practice, no one, after doing a single day's work according to Ritter, would need to write out any equations as fully as done here, and for most members would not write them out at all, unless indeed "in his mind's eye."

$$S_1 \times 172.01 - 14.17 \times 222 = 0.$$
$$-T_2 \times 184.02 - 14.17 \times 222 + 23.4 \times (222 + 18) = 0.$$
$$S_3 \times 211.62 - 14.17 \times 222 + 23.4 \times (222 + 18) = 0.$$
$$-T_4 \times 222.43 - 14.17 \times 222 + 23.4 \times [(222 + 18) + (222 + (3 \times 18))] = 0.$$
$$S_5 \times 251.44 - 14.17 \times 222 + 23.4 \times [(222 + 18) + (222 + (3 \times 18))] = 0.$$

$$-P_1 = A.$$
$$-P_2 \times [222 + (5 \times 18)] - (57.33 - 24.7) \times [222 + (5 \times 18)] - 14.17 \times 222 + 23.4 \times [444 + (4 \times 18)] = 0.$$

$T_6 \sin \phi \infty - 14.17 \times \infty + 23.4 \times \infty + 23.4 \times \infty - (57.33 - 24.7) \times \infty = 0$, where ϕ is the angle T_6 makes with the horizontal.

$$\therefore \text{Sin } \phi = \frac{25}{32.803}, \text{ and dividing by } \infty:$$

$$T_6 \times \frac{26}{32.803} - 14.17 + 23.4 + 23.4 - 32.63 = 0.$$

SECOND SYSTEM.

$$U_{2\,\&\,3} \times 21.425 + (15.35 - 11.7) \times (2 \times 18) = 0.$$
$$U_{4\,\&\,5} \times 24.414 + 3.65 \times (4 \times 18) - 23.4 \times (2 \times 18) = 0.$$
$$U_6 \times 26 + 3.65 \times [(5 \times 18) + 20] + 43.15 \times 20 - 23.4 \, [((3 \times 18) + 20) + (18 + 20)] = 0.$$

$$-L_{1\,\&\,2} \times 20 + (15.35 - 11.7) \times 18 = 0.$$
$$-L_{3\,\&\,4} \times 23 + 3.65 \times (3 \times 18) - 23.4 \times 18 = 0.$$
$$-L_5 \times 26 + 3.65 \times (5 \times 18) - 23.4 \, [(3 \times 18) + 18] = 0.$$
$$-L_6 \times 26 + 3.65 \times [(5 \times 18) + 20] - 23.4 \, [((3 \times 18) + 20) + (18 + 20)] + 43.15 \times 20 = 0.$$

$-T_1 \times 165.01 - 3.65 \times 222. = 0$ [or, revolving about second load point: $+T_1 \times \sqrt{36^2 - 26.907^2} + 3.65 \times 36 = 0$].

$$S_2 \times 191.77 - 3.65 \times 222 = 0.$$
$$-T_3 \times 203.17 - 3.65 \times 222 + 23.4 \times [222 + (2 \times 18)] = 0.$$
$$S_4 \times 231.52 - 3.65 \times 222 + 23.4 \times [222 + (2 \times 18)] = 0.$$
$$-T_5 \times 241.72 - 3.65 \times 222 + 23.4 \times [(222 + (2 \times 18)) + (222 + (1 \times 18))] = 0.$$

$$-P_2 \times [222 + (5 \times 18)] - 3.65 \times 222 + 23.4 \, [444 + (6 \times 18)] - T_5 \times 241.72 - 43.15 \times [222 + (5 \times 18)] = 0.$$

$$T_6 \sin \phi - 3.65 + 23.4 + 23.4 - 43.15 = 0.$$

The other half girder must be made symmetrical, or, if desired (in the latched cases), can be calculated, either by continuing to make the supposed sections in the order of procedure from left to right, always introducing reactions as though they were merely minus loads (which *is* all they are), or by taking sections anew in the order of from right to left.

In this way have been found the strains in Plates III.–IX., and tabulated in Table (*A*), p. 444. Observe that the centre panel diagonals present a peculiar feature: the diagonal in service is a different one for the cases (*a*) and (*b*), and has a different value for the two, but each diagonal has the same value whether in *III.*, *IV.*, or *V.* $A + B$ is constant, in pivoted as well as fixed centre support draws:

$$\text{In the first,} \quad A + B < \text{loads on } AB.$$
$$\text{In the second,} \quad A + B > \text{loads on } AB.$$

A careful study of this table will prove instructive also as to choice of *kind* of supports for drawbridges. The table is not complete, however; there should be added the strains for the cases where there is no snow load, especially for *Case IV.* (*b*), and in the span *CD*. These may be obtained in two ways: (1) we could get the new reactions, by subtracting, for each support, the reactions produced there by the snow load from those already found, and then proceeding in the calculation of strains as before, or (2) by finding first the strains produced in each member by the snow load, and then algebraically subtracting these strains from those already found in the corresponding members. The snow strains are, however, of two kinds; once when the weight of the snow is all carried by the centre supports, and again when the truss carrying the snow loads rests on 4 level supports. The first mentioned snow strains obtain in *Cases I.* and *III.* (*b*), and must also be subtracted to get the strains without snow in *Cases V.* (*a*) and (*b*), whenever the end reactions without snow are to be the same they were with snow. (In *Cases V.* (*a*) and (*b*), it will be remembered, the end supports are lifted, just enough to prevent the end reactions from ever becoming less than a chosen plus quantity.) For, the end reactions remaining the same, the centre reactions only can have diminished, and must have diminished by just the weight of the snow; and the strains can have diminished algebraically, only by the strains produced by these two centre reactions, together equal to the snow load, acting as opposed to the several panel snow loads. Another view would be this: we had made *s*, the part of the own weight that was carried on the 4 supports, instead of on only the central ones, just large enough to securely keep the light end from rising in the event of maximum unsymmetrical loading. If now the snow melts off, it will render the light end insecure, and enough additional *s* must be taken out of the own weight to restore the original end reactions; that is, the new *s* must = old *s* + snow. The process is, the snow is removed from the 4 level supports, then a weight equal to it is taken off of the centre

supports, and is distributed over the 4 level supports again, leaving, as a net result, the centre supports lighter, by the total snow load. Mechanically, this will necessitate a higher *position* of the end supports, at the end of the lift, in order that the same end reactions may be obtained without, as before were obtained with, the snow. Were, on the contrary, the end supports to remain in the same position, or at the same level, then, to get the strains in the trusses without the snow load, from those above with the snow on, we should have to subtract the snow strains produced by the snow load resting on 4 level supports.

Figs. 20 and 21, Plate X., show the two kinds of snow strains; Fig. 20, being those when the snow reactions are produced only at the centre supports, and are evidently simply $\frac{1}{6}$ the strains given for *Case I.* The others (Fig. 21) are the strains due a load of 200 lbs. per foot on a bridge skeleton without weight, and of the form under consideration, and supported on 4 level supports. Now subtract these strains in the several members, algebraically, from the corresponding strains in the table, remembering which set to use in the several cases, as above given. Fig. 20 has been used in *Case V.* (*b*) and Fig. 21 in *Case V.* (*a*), since $\frac{1}{3}$ dead was more than enough for s in this latter case, and $D = +$, even without the snow.

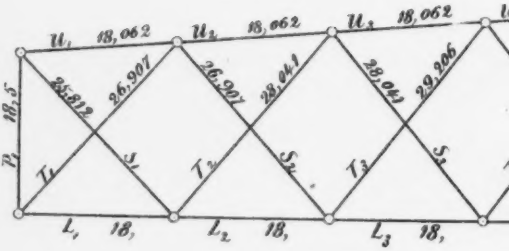
The resultant strains for all the cases will be found in Table (*B*), on p. 445.

While this article is going through the press, the author notices, in a work* recently issued, the following sentence: "But no book in common use gives us any method for determining the shearing stress under a partial load, a determination which is necessary before the bracing can be correctly proportioned." It is to be hoped, however, that no insuperable objection lies in the way to the attainment of such an object, and that such a book may be written. The following has been an attempt in that direction; for the careful reader will have noticed that, by the method and the formulæ herein given, the effect of a load at any point or points, on a continuous girder, upon any of its members, may be calculated.

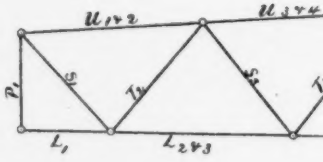
The method of strain calculation followed throughout has been what may be termed the "arithmetical" one; and the author has purposely presented the same purely and alone, and omitting, therefore, considerations of mo-

* Graphical Method for the Analysis of Bridge Trusses, &c. Charles E. Greene. New York. 1875. Van Nostrand.

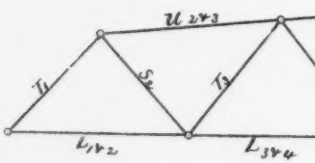
Names and length



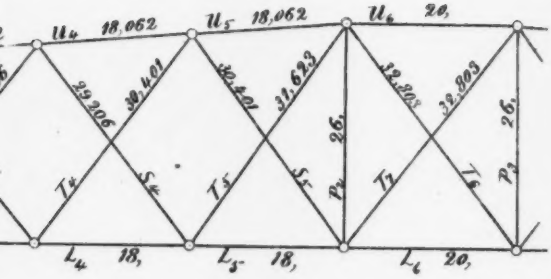
First S



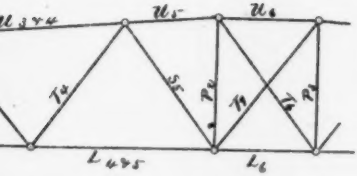
Second



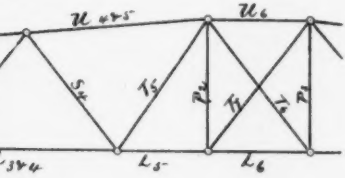
ngths of the Members.



rst System.



cond System.



Calculation of the lever-arms.

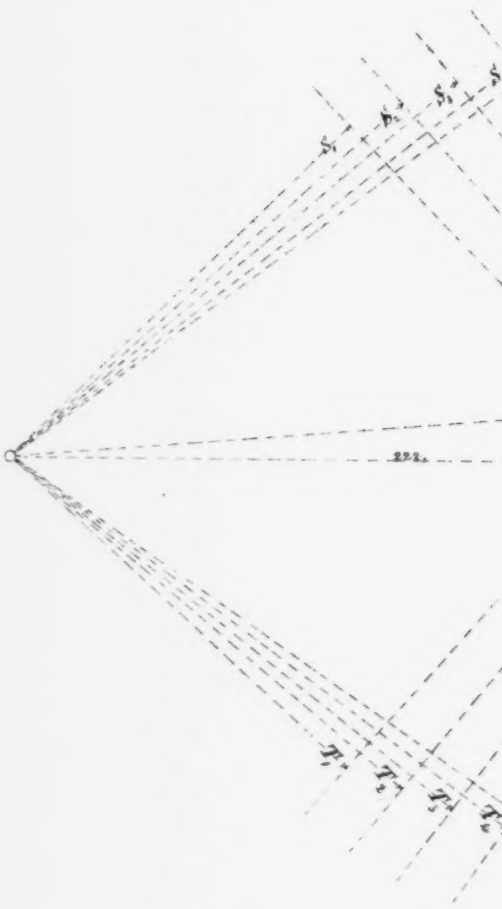


PLATE II.

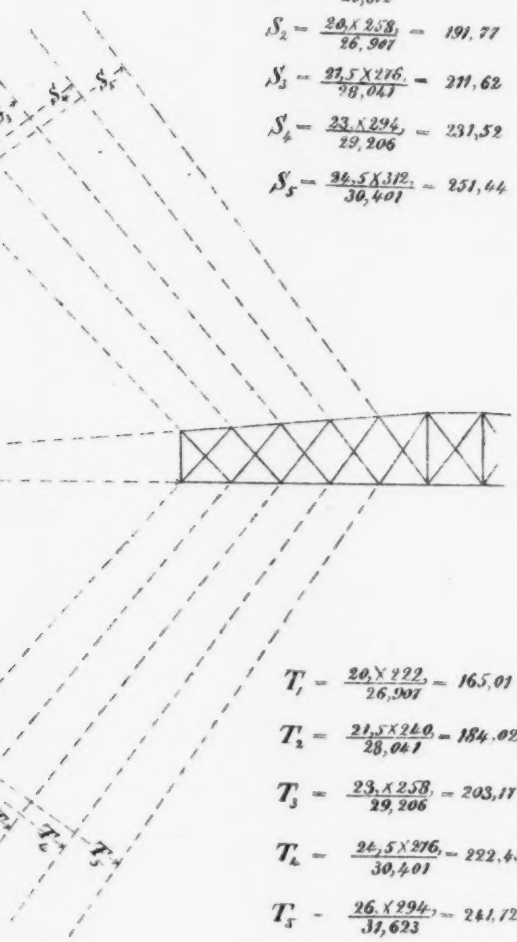
$$S_1 = \frac{18,5 \times 240,}{25,312} = 172,005$$

$$S_2 = \frac{20,\times 258,}{26,901} = 191,77$$

$$S_3 = \frac{22,5 \times 276,}{28,041} = 211,62$$

$$S_4 = \frac{23,\times 294,}{29,206} = 231,52$$

$$S_5 = \frac{24,5 \times 312,}{30,401} = 251,44$$



$$T_1 = \frac{20,\times 222,}{26,901} = 165,01$$

$$T_2 = \frac{21,5 \times 240,}{28,041} = 184,02$$

$$T_3 = \frac{23,\times 258,}{29,206} = 203,17$$

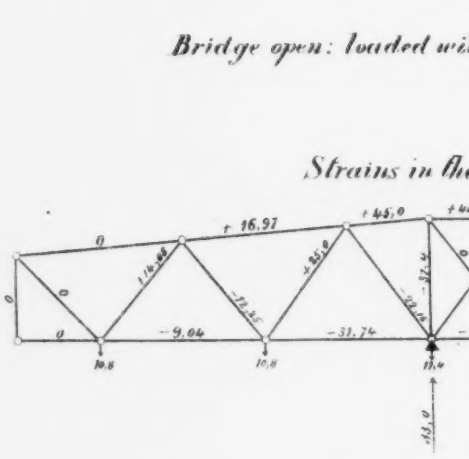
$$T_4 = \frac{24,5 \times 276,}{30,401} = 222,43$$

$$T_5 = \frac{26,\times 294,}{31,623} = 241,72$$

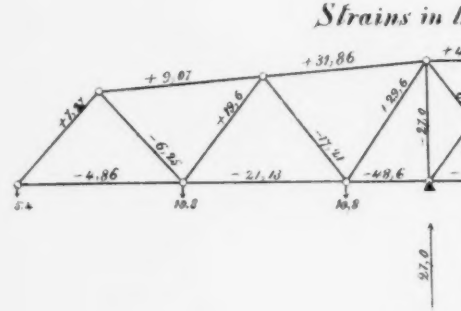
Case I

Bridge open: loaded wi

Strains in the



Strains in t



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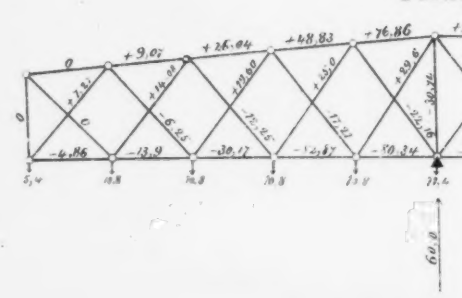
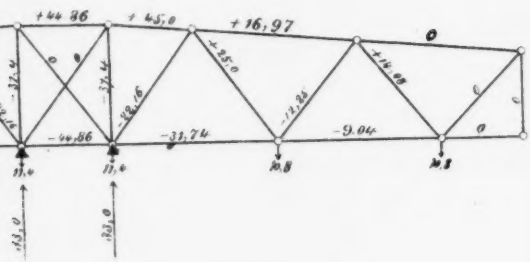


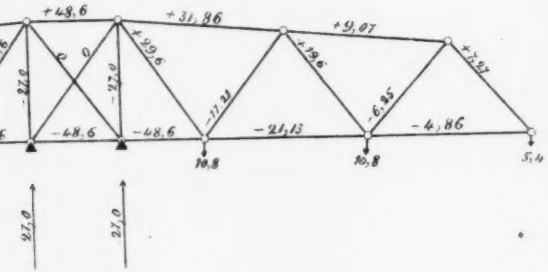
e I.

d with dead + snow.

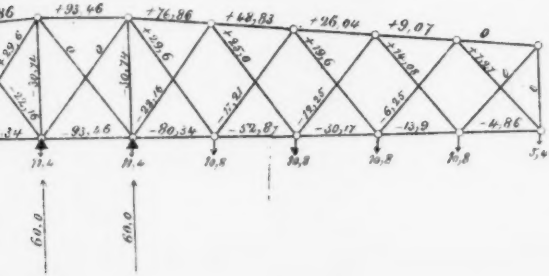
in the first System.



s in the second System.



Total Strains.

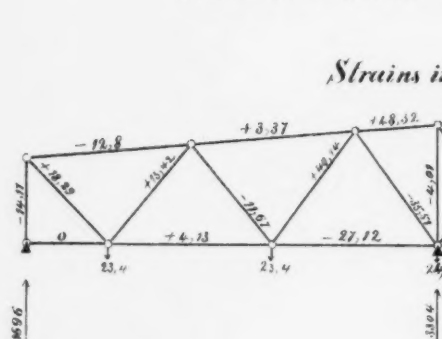


Case

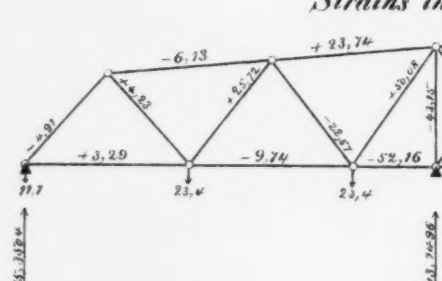
Bridge closed : loaded wi

ends due to snow + live

Strains i



Strains i



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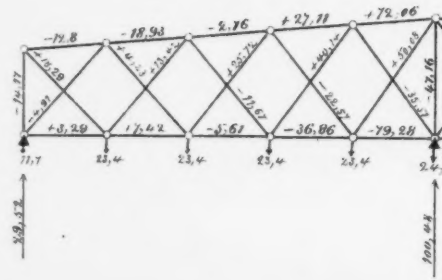
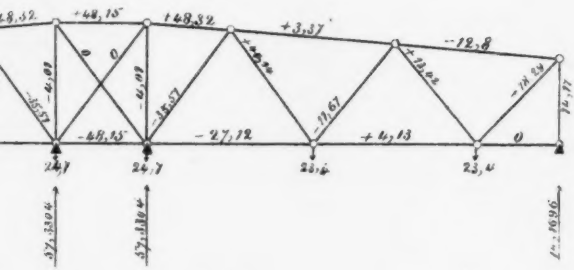


ase II.

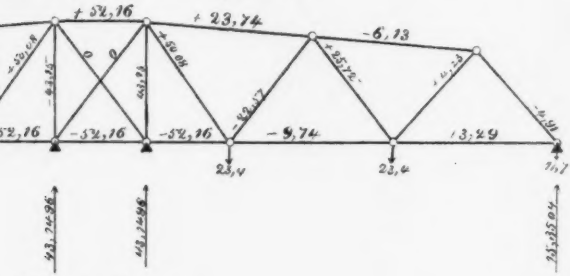
led with dead + snow + live ; reactions at the

live only.

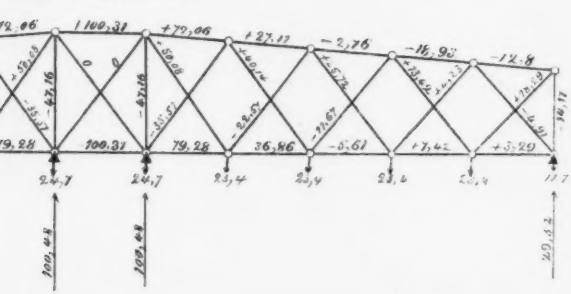
ins in the first System.



ns in the second System.



Total Strains.



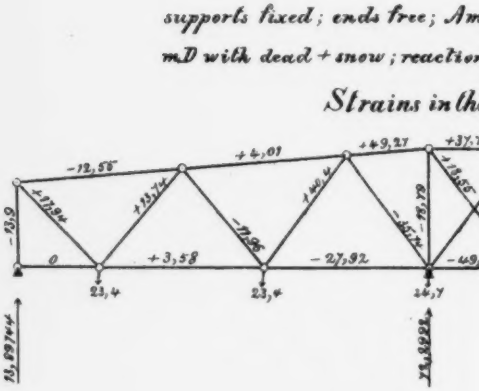
Case I

[Also Case IIIa) in this exampl

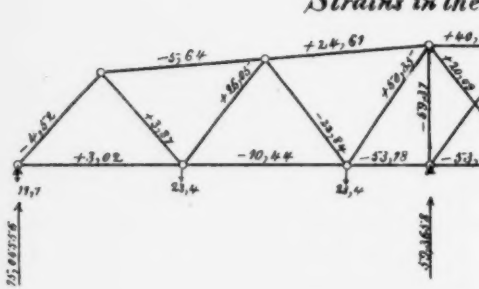
supports fixed; ends free; Am

mD with dead + snow; reaction

Strains in th



Strains in the



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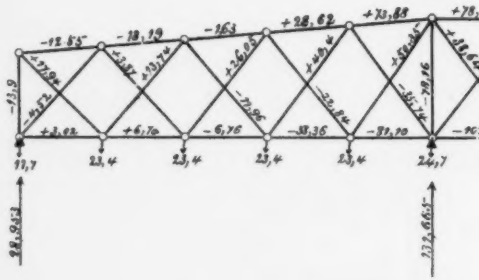
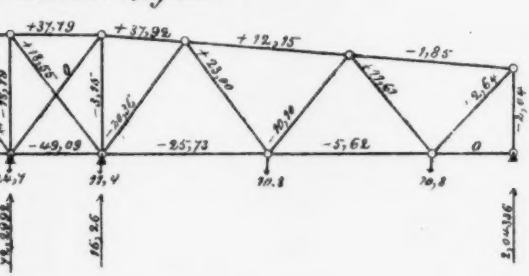
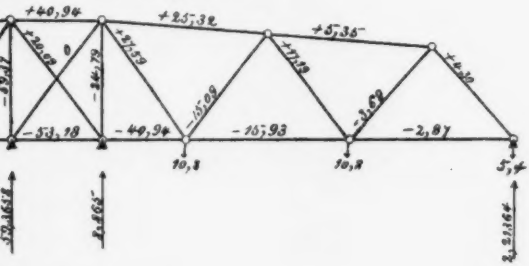


...se III(a).

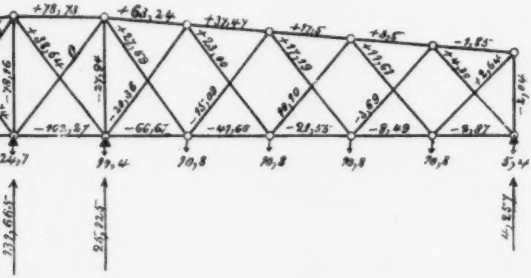
...xample, because D is plus / Centre
...e; Am loaded with dead + snow + live;
...actions at ends due to snow + live only
... in the first System.



... in the second System.



...otal Strains.

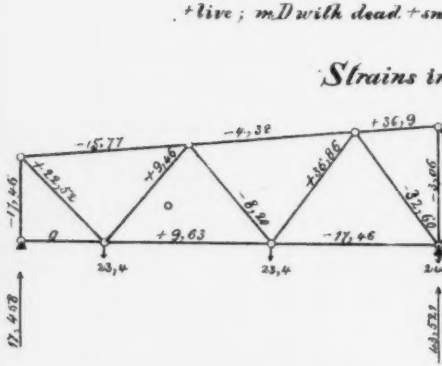


Cas

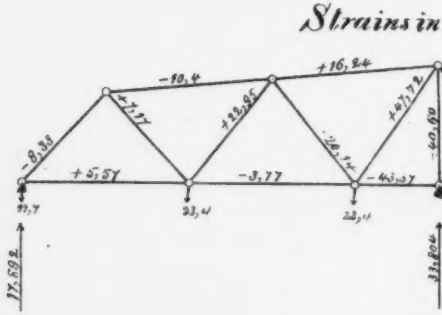
Centre supports pivotted; e

+ live ; m D with dead + sn

Strains i



Strains in



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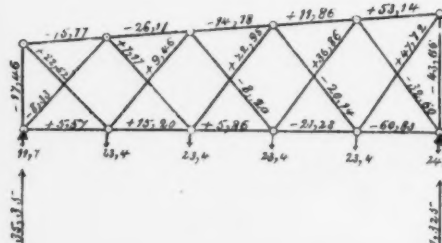
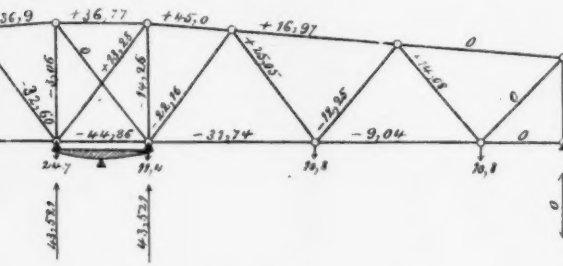


Case III(h).

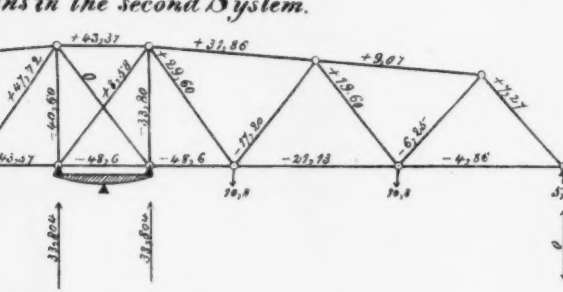
tted; ends free. Am loaded with dead + snow

d. + snow; reactions at ends due to snow + live only.

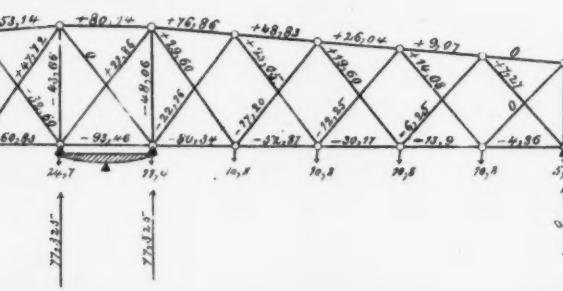
ins in the first System.



ins in the second System.



Total Strains.

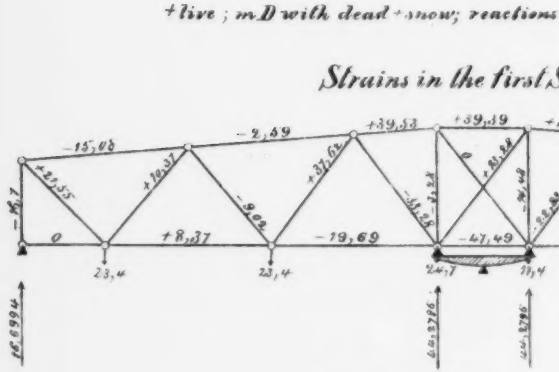


Case IV(b)

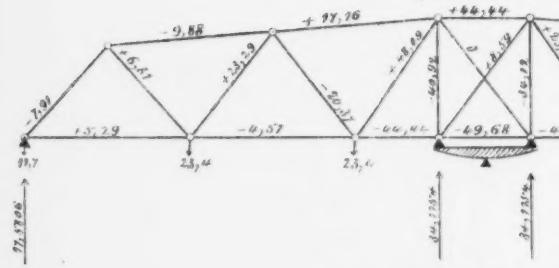
Centre supports pivotted: ends latched

+live ; mD with dead + snow; reactions

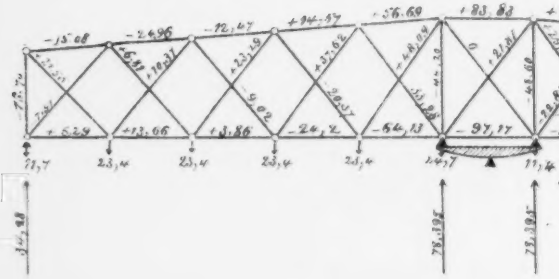
Strains in the first S



Strains in the secon



Total Strai

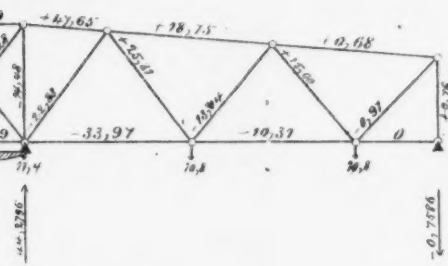


IV(b).

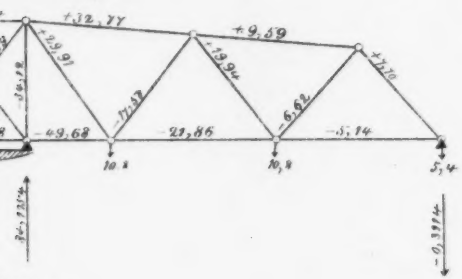
tched ; Am loaded with dead + snow

ctions at ends due to snow + live only.

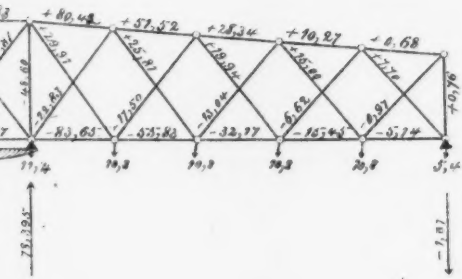
irst System.



econd System.



trains.

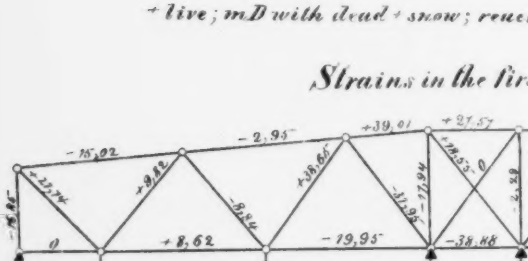


Case V(a

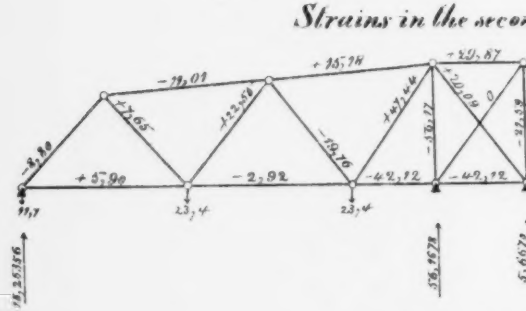
Centre supports fixed; ends lifted

+ live; nD with dead + snow; reac

Strains in the fir



Strains in the secon



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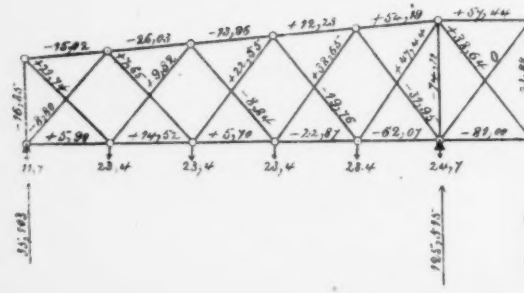
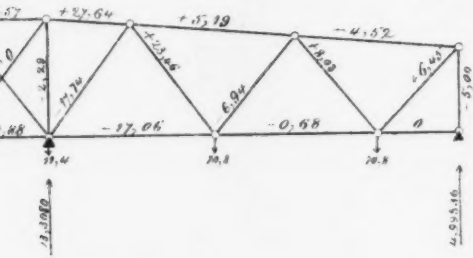


V(a).

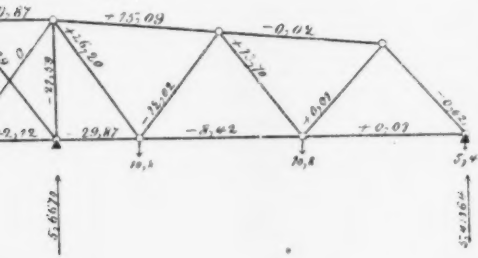
lifted: Am. loaded with dead + snow

reactions at ends due to live + snow + ⅓ dead.

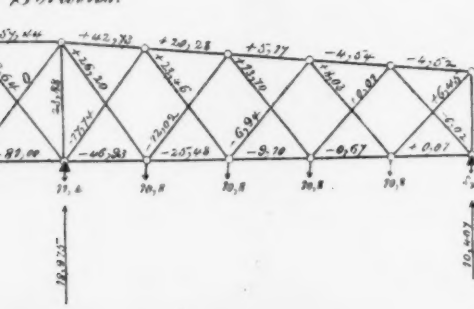
e first System.



second System.



Strains.

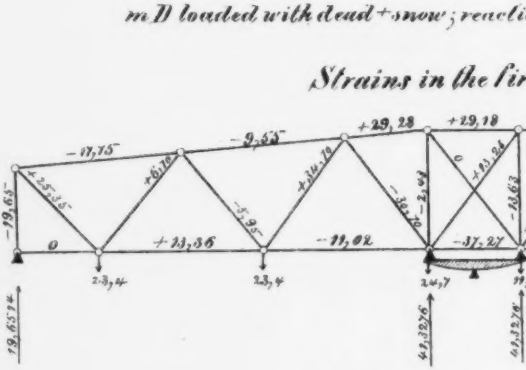


Case V(b).

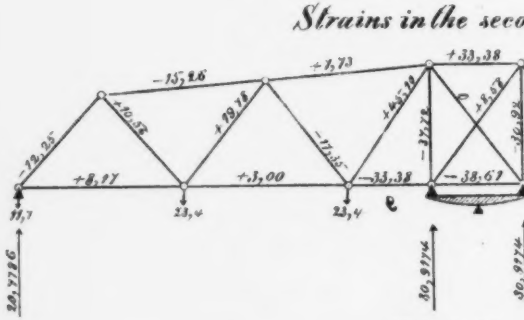
Centre supports pivotted; ends lifted;

m D loaded with dead + snow; reacti

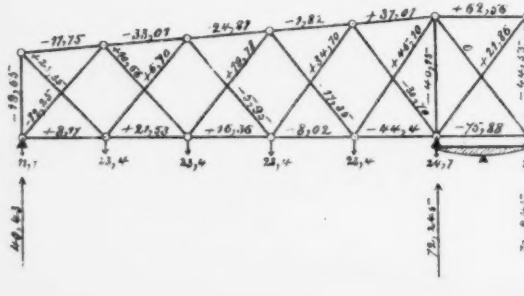
Strains in the fir



Strains in the seco



Total, Stra

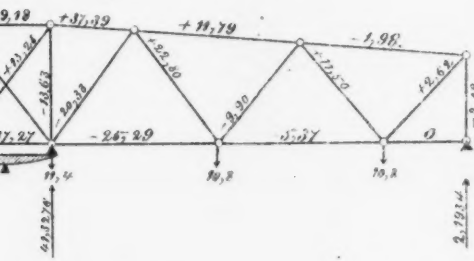


Vb).

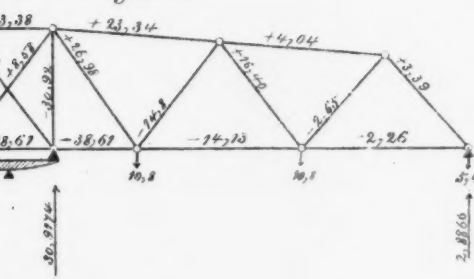
illed; Am loaded with dead + snow + live;

reactions at ends due to snow + live + ⅓ dead.

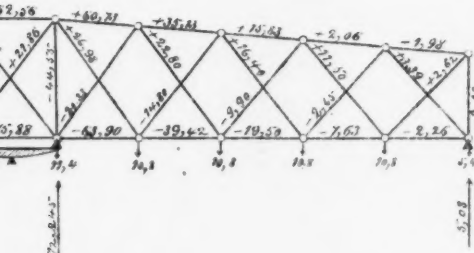
he first System.



second System.



Strains.



Strains produced

Fig. 20

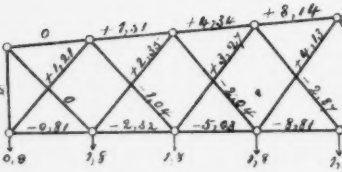
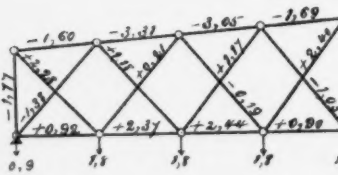
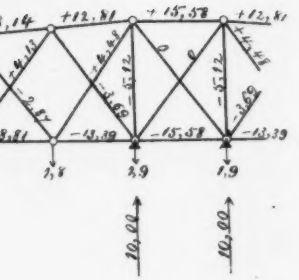


Fig. 2

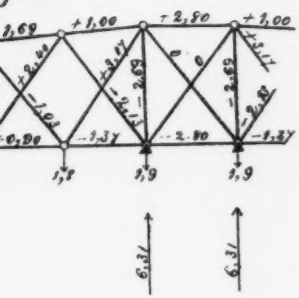


ced by the snow-load.

g. 20.



g. 21.



ment-curves, of vertical and horizontal shears, points of contrary flexure, and other paraphernalia which usually accompany treatises on continuous girders.

To calculate the absolute maxima and minima strains in the members of a continuous panel girder by the method that has been followed, the process would be like this : —

First find the strains in each member, under the own weight of the bridge.

Next find the strains in each member for the maximum panel live load on the first panel point, all the rest of the bridge being supposed to be without weight and unloaded.

Next place the panel live load on the second panel point, the rest of the bridge being supposed to be without weight and unloaded, and so on, for each panel point. By tabulating the strains thus found for each member of the bridge, a clear oversight will be gained over them. Then summing up all the minus strains liable to occur in any one member, with the strain due to the own weight, will give the maximum minus strain, and summing up its plus strains with the same will give the maximum plus strain that can come upon it by any possible combination of loads on the bridge.

TABLE (*A*) OF STRAINS.

Pieces.	*Case I.*	*Case II.*	*Case III.* (*a*)		*Case III.* (*b*)		*Case IV.* (*a*)		*Case IV.* (*b*)		*Case V.* (*a*)		*Case V.* (*b*)	
			Span *AB.*	Span *CD.*	Span *AB.*	Span *CD.*	Span *AB.*	Span *CD.*	Span *AB*	Span *CD.*	Span *AB.*	Span *CD.*	Span *AB.*	Span *CD.*
U_1	0.	−12 8	−12.6	− 1.9	−15.8	0.	−12.6	− 1.9	−15.1	**+ 0.7**	−15 0	− 4.5	**−17.8**	− 2.0
U_2	+ 9.1	−18.9	−18.2	+ 3.5	−26.2	+ 9 1	−18.2	+ 3.5	−25.0	**+10.3**	−26.0	− 4.5	**−33.0**	+ 2 1
U_3	+26.0	− 2.8	− 1.6	+17.5	−14.8	+26.0	− 1.6	+17.5	−12.5	**+28.3**	−14.0	+ 5.2	**−24.8**	+15.8
U_4	+48.8	+27.1	+28.6	+37.5	+11.9	+48.8	+28.6	+37.5	+14.6	**+51.5**	+12.2	+20.3	**− 1.8**	+35.1
U_5	+76.9	+72.1	+73.9	+63.2	+53.1	+76.9	+73 9	+63.2	+56.7	**+80.4**	+51.2	+42.7	+ 37.0	+60.7
U_6	+93.5	**+100.3**	+78	.7	+80	.1	+78	.7	+83	.8	+57	.4	+62	.6
L_1	− 4.9	+ 3.3	+ 3.0	− 2.9	+ 5.6	− 4.9	+ 3.0	− 2.9	+ 5.3	**− 5.1**	+ 5.9	0.	**+ 8.2**	− 2.3
L_2	−13.9	+ 7.4	+ 6.7	− 8.5	+15.2	−13.9	+ 6.7	− 8.5	+13.7	**−15.5**	+14.5	− 0.7	**+21.5**	− 7.6
L_3	−30.2	− 5.6	− 6.8	−21.6	+ 5.9	−30.2	− 6.8	−21.6	+ 3.9	**−32.2**	+ 5.7	− 9.1	**+16.4**	−19.5
L_4	−52.9	− 36 9	−38.4	−41.7	−21.2	−52.9	−38.4	−41.7	−24.2	**−55.8**	−22.9	−25.5	− 8.0	−39.4
L_5	−80.3	− 79.3	−81.1	−66.7	−60.8	−80.3	−81.1	−66.7	−64.1	**−83.7**	−62.1	−46.9	**−44.4**	−63.9
L_6	−93.5	−100.3	**−102**	.3	−93	.5	**−102**	.3	−97	.2	−81	.0	−75	.9
P_1	0.	−14.2	−13.9	− 2.0	−17.5	0.	−13.9	− 2.0	−16.7	**+ 0.8**	−16.9	− 5.0	**−19.7**	− 2.2
P_2	−30.7	−47.2	**−78.2**	−27.9	−43.7	−48.1	**−78.2**	−27.9	−44.2	−48.6	−74.1	−23.9	−40.2	−44.6
T_1	+ 7.3	− 4.9	− 4 5	+ 4.3	− 8.3	+ 7.3	− 4 5	+ 4.3	− 7.9	**+ 7.7**	− 8.8	0.	**−12.3**	+ 3.4
T_1'	+14.1	+13.4	+ 13.7	+11.6	+ 9.5	+14.1	+ 13.7	+11.6	+10.4	**+15.0**	+ 9.8	+ 8.0	+ 6 7	+11.5
T_2	+19 6	+25.7	**+26.1**	+17.2	+23.0	+19.6	**+26.1**	+17.2	+23.3	+ 19.9	+22.6	+13.7	+ 19.8	+16.4
T_3	+25.0	+40.1	**+40.4**	+23.0	+36.9	+25.1	**+40.4**	+23.0	+37.6	+ 25.8	+38.7	+23.5	+ 34.7	+22.8
T_4	+29.6	+50.1	**+50.4**	+27.6	+47.7	+29.6	**+50.4**	+27.6	+48.1	+ 29.9	+47.4	+26.2	+ 45.1	+27.0
T_5	0.	0.	0	.	**+21**	.9	0	.	**+21**	.9	0	.	**+21**	.9
T_6	0.	0.	**+38**	.6	0	.	**+38**	.6	0	.	**+38**	.6	0	.
S_1	0.	+18.3	+ 17.9	+ 2.6	+22.5	0.	+ 17.9	+ 2.6	+21.6	**− 1.0**	+21.7	+ 6.5	**+25.4**	+ 2.6
S_2	− 6.3	+ 4.2	+ 3.9	− 3.7	+ 7.2	− 6.3	+ 3.9	− 3.7	+ 6 8	**− 6.6**	+ 7.7	0.	**+10.6**	− 2.7
S_3	−12.3	−11.7	−12.0	−10.1	− 8.2	−12.3	−12.0	−10.1	− 9.0	**−13.0**	− 8.8	− 6.9	− 6.0	− 9.9
S_4	−17.2	−22.6	**−22.8**	−15.1	−20.1	−17.2	**−22.8**	−15.1	−20.4	− 17.5	−19.8	−12.0	− 17.4	−14.8
S_5	−22.2	−35 5	**−35.7**	−20.4	−32 6	−22.2	**−35.7**	−20.4	−33.3	− 22.8	−32 0	−17.7	− 30.7	−20.3

The figures in heavy type show the maximum strains in the several members.

TABLE (*B*) OF STRAINS WITH THE SNOW LOAD REMOVED.

Pieces.	*Case I.*	*Case II.*	*Case III.(a)*		*Case III.(b)*		*Case IV.(a)*		*Case IV.(b)*		*Case V.(a)*		*Case V.(b)*	
			Span *AB.*	Span *CD.*	Span *AB.*	Span *CD.*	Span *AB.*	Span *CD.*	Span *AB.*	Span *CD.*	Span *AB.*	Span *CD.*	Span *AB.*	Span *CD.*
U_1	0.	−11.2	−11.0	− 0.3	−15.8	0.	−11.0	− 0.3	−13.5	**+ 2.3**	−13.4	− 2.9	−17.8	− 2.0
U_2	+ 7.6	−15.6	−14.9	**+ 6.8**	**−27.7**	+ 7.6	−14.9	**+ 6.8**	−21.7	**+13.6**	−22.7	− 1.2	**−34.5**	+ 0.6
U_3	+21.7	**+ 0.3**	**+ 1.4**	**+20.6**	−19.1	+21.7	**+ 1.4**	**+20.6**	− 9.4	**+31.4**	−10.9	+ 8.2	**−29.1**	+11.5
U_4	+40.7	**+28.8**	**+30.3**	+39.2	+ 3.7	+40.7	**+30.3**	+39.2	+16.3	**+53.2**	**+13.9**	**+22.0**	**− 9.9**	+27.0
U_5	+64.1	+71.1	+72.9	+62.2	+40.3	+64.1	+72.9	+62.2	+55.7	+79.4	+53.2	+41.7	+24.2	+47.9
U_6	+77.9	+97.5	+75.9		+64.6		+75.9		+81.0		+54.6		+47.0	
L_1	− 4.1	+ 2.4	+ 2.1	**− 3.8**	+ 6.4	− 4.1	+ 2.1	**− 3.8**	+ 4.4	**− 6.1**	+ 5.0	**− 0.9**	+ 9.0	− 1.5
L_2	−11.6	+ 5.1	+ 4.3	**−10.9**	+17.5	−11.4	+ 4.3	**−10.9**	+11.3	**−17.8**	+12.2	**− 3.0**	**+23.8**	− 5.3
L_3	−25.1	**− 8.1**	**− 9.2**	**−24.0**	+10.9	−25.1	**− 9.2**	**−24.0**	+ 1.4	**−34.6**	+ 3.3	**−11.5**	**+21.4**	−14.5
L_4	−44.1	**−37.8**	**−39.3**	**−42.6**	−12.4	−44.1	**−39.3**	**−42.6**	**−25.1**	**−56.7**	**−23.8**	**−26.4**	+ 0.8	−30.6
L_5	−67.0	−77.9	−79.7	−65.3	−47.4	−67.0	−79.7	− 65.3	−62.8	−82.3	− 60.7	−45.6	−31.0	−50.5
L_6	−77.9	−97.5	−99.5		−77.9		−99.5		−94.4		−78.2		−60.3	
P_1	0.	−12.4	−12.1	− 0.3	−17.5	0.	−12.1	− 0.3	−14.9	**+ 2.5**	−15.1	− 3.2	−19.7	− 2.2
P_2	−25.7	−44.5	−75.5	−25.3	−38.5	−42.9	−75.5	−25.3	−41.5	−45.9	−71.4	−21.2	−35.1	−39.5
T_1	+ 6.1	− 3.5	− 3.1	**+ 5.7**	**− 9.5**	+ 6.1	− 3.1	**+ 5.7**	− 6.5	**+ 9.1**	− 7.4	**+ 1.4**	**−13.5**	+ 2.2
T_2	+11.7	+13.2	+13.5	+11.4	+ 7.1	+11.7	+13.5	+11.4	+10.2	+14.8	+ 9.6	+ 7.8	+ 4.3	+ 9.1
T_3	+16.3	+24.6	+24.9	+16.0	+19.7	+16.3	+24.9	+16.0	+22.1	+18.8	+21.4	+12.5	+16.5	+13.1
T_4	+20.9	+37.7	+38.0	+20.6	+32.7	+20.9	+38.0	+20.6	+35.2	+23.4	+36.3	+21.1	+30.5	+18.6
T_5	+24.7	+46.9	+47.2	+24.4	+42.8	+24.8	+47.2	+24.4	+44.9	+26.8	+44.3	+25.0	+40.6	+22.5
T_6	0.	0.	0.		+21.9		0.		+21.9		0.		+21.9	
T_7	0.	0.	+38.6		0.		+38.6		0.		+38.6		0.	
S_1	0.	+16.0	+15.7	+ 0.4	+22.5	0.	+15.7	+ 0.4	+19.3	**− 3.3**	+19.5	+ 4.2	+25.4	+ 2.6
S_2	− 5.2	+ 3.1	+ 2.7	− 4.9	+ 8.2	− 5.2	+ 2.7	**− 4.9**	+ 5.6	**− 7.8**	+ 6.5	**− 1.2**	**+11.6**	− 1.7
S_3	−10.2	−11.5	−11.8	− 9.9	− 6.2	−10.2	−11.8	− 9.9	− 8.8	−12.9	− 8.7	− 6.8	− 4.0	− 7.9
S_4	−14.3	−21.5	−21.8	−14.1	−17.3	−14.3	−21.8	−14.1	−19.3	−16.5	−18.7	−11.0	−14.5	−11.9
S_5	−18.5	−33.4	−33.6	−18.2	−28.9	−18.5	−33.6	−18.2	−31.2	−20.7	−29.8	−15.6	−27.0	−16.6

☞ The figures in heavy type show the maximum strains in the several members, as compared with the strains of the previous table.

APPENDIX.

LIST OF BOOKS AND ARTICLES IN TECHNICAL JOURNALS, RELATING TO THE SUBJECT OF CONTINUOUS GIRDERS (IN PART OR IN WHOLE), IN THE GERMAN, FRENCH, AND ENGLISH LANGUAGES, 1854–1874.

[Without wishing to presume the part of an adviser, the writer yet offers the opinion that the starred works are especially to be recommended to those desirous of further pursuing the subject of Continuous Girders. The annexed list is doubtless incomplete.]

**Weyrauch, Dr. Phil. Jacob J.*, Ingenieur, &c. Allgemeine Theorie und Berechnung der Continuirlichen und Einfachen Träger. Leipzig: B. G. Teubner. 1873.

**Ritter, August*, Prof. &c. Elementare Theorie und Berechnung eiserner Dach- & Brücken-Constructionen. (Contains, amongst other matters, a peculiar *kind* of continuous girder.) Hannover: Carl Rümpler. 1873.

**Laissle & Schuebler.* Der Bau der Brücken-Träger. Stuttgart: Paul Neff. 2 vols. 1869 & 1870. (This work has also appeared in a French translation.)

Winkler, Dr. E. Elasticität und Festigkeit. Prag: H. Dominicus. 1867.

Zeitschrift des Archt. & Ing. Verein zu Hannover.

**Mohr* (now Prof. &c. in Stuttgart) . . . . . 1860, 1862, & 1868.
Köpke, C., Ingr. &c. . . . . 1856 & 1857.

Zeitschrift für Bauwesen, Berlin.

Quensel, O. . . . . . 1865.
Schwedler, J. W. . . . . . 1862.
Winkler, E. O. . . . . . 1860.

Zeitschrift des Oestr. Archt. & Ing. Verein.

Winkler, Prof. &c. . . . . 1872.

Der Civil Ingenieur.

Fränkel, W. . . . . . 1868, p. 271.
—— —— . . . . . 1860, Heft 3 & 4.
Hallbauer . . . . . 1858, ,, 5 & 6.
—— —— . . . . . 1858, ,, 2.

ZEITSCHRIFT DES VEREINS DEUTSCHER INGENIEURE.

Grashof . . . . . 1857.

ZEITSCHRIFT DES BAYERISCHEN ARCHT. & ING. VEREIN.

Gerber. (A Patent Continuous Girder.) . . . . . 1870.

ALLGEMEINE BAUZEITUNG.

Schmidt, Heinrich . . . . . 1868.

De Montdesir. (Strains in Continuous Trusses of Iron Bridges.) 2d Ed. . . . . 1873.

**Bresse.* Cours de Mécanique Appliquée. 1re & 3me partie. Paris. 1859 & 1865.

ANNALES DES PONTS ET CHAUSÉES.

**Renaudot* . . . . . 1866, p. 311.

Pierre . . . . . 1871, p. 44.

Albaret . . . . . 1866, p. 53.

"COMPTES RENDUS."

Clapeyron . . . . . Dec. 1857.

Molinos & Pronnier. Construction des Ponts Métalliques. Paris. 1857.

ANNALES DES TRAVAUX PUBLICS DE BELGIQUE.

De Clerq . . . . . 1855-56.

[*Navier.* Résumé des Leçons données à l'École des P. & C. &c. 2d Ed. Paris. 1833.]

MINUTES OF THE PROCEEDINGS OF THE INST. CIV. ENGRS.

Bell, W. . . . . . Vol. 32, p. 171.

**Stoney, E. W.* . . . . . ,, 29, p. 382.

**Heppel, J. M.* . . . . . ,, 19, p. 625.

**Barton, James* . . . . . ,, 14, p. 443.

Contains a description of the celebrated, and, considering the time it was built, *wonderfully perfect*, Boyne Bridge at Drogheda, Ireland, 3 continuous spans of 141, 267, and 141 feet, opened for traffic, double track R.R., in April, 1855.

Pole, W. . . . . . Vol. 9, p. 261.

Hemans, G. W. . . . . . ,, 3, p. 65.

THE PHILOSOPHICAL MAGAZINE, LONDON.

**Heppel, J. M.* . . . . . Vol. 40, p. 446.

Stoney, B. B. Theory of Strains in Girders, &c. London: Longmans, Green, & Co. . . . . 1873.

Rankine, W. J. M. Civil Engineering . . . . . pp. 287–292.

Humber, William. Strains in Girders. Am. Ed. New York: Van Nostrand . . . . . 1870.

Clark, Edwin. Brittania and Conway Tubular Bridges. London. 1850.
Moseley, Henry. Mechanical Principles of Engineering and Architecture (1840–42). Am. Ed. By D. H. Mahan, Professor at West Point. New York: J. Wiley & Son. 1869. (Continuous beams are treated of in this work by the name of "Breast summers.")
Chanute, O., and *G. Morison.* Kansas City Bridge. pp. 86, 87, 108–114.

JOURNAL OF THE FRANKLIN INSTITUTE.

Frizell, J. P. . . . . . 3d Series, Vol. 64, 1872.

TRANSACTIONS ENGINEERS' CLUB OF ST. LOUIS.

Smith, Charles A. Girders Continuous over Supports . . Dec. 4, 1872.

VAN NOSTRAND'S ECLECTIC ENGINEERING MAGAZINE.

Eddy, C. H. . . . . . p. 352, 1874.

DUTCH LANGUAGE.

Tijdschrift van het Koninklijk Instituut van Ingenieurs.* 's Gravenhage . . . . . 1870 & 1871.

* The writer does not know the exact nature of these articles.